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Paper 1042 PO 4

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THE EFFECT OF PORE PRESSURES ON STRESSES IN GRAVITY DAMS

O. C. Zienkiewicz[1]
(Proc. Paper 1042)

SYNOPSIS

In this paper the question of stresses arising in a gravity dam due to the action of pore water pressures is considered. Although for a particular type of pressure distribution the answer to this problem has been derived, the author seeks to provide a solution for cases, often existing in practice, where the restrictive assumptions of the known solution do not apply.

In particular, a case of an idealised, triangular shaped dam is analysed, and a general elastic solution obtained. This solution is valid for any arbitrary type of pressure variation. Numerical answers are given to a special case, where a line of drains placed inside the dam reduces the pore pressures discontinuously. Some simple general conclusions are drawn from the results of above computations.

INTRODUCTION

The voluminous literature and considerable amount of controversy on the subject of pore pressure effects, have, in greater part, been concerned with the problems of effective uplift area, i.e. the proportion of the cross-sectional area of the material on which the pore pressures may be taken as active. It is not the intention of the author to add any further contribution to this matter and, though a value of the above factor close to unity is believed to be correct, the arguments presented here are equally applicable to any other value of this coefficient chosen.

The main concern of this work is the important question: How do the internal pore pressures affect the stress distribution in a dam? One of the most notable contributions to the answer of this question was given by J. H. Brahtz at the International Congress on Large Dams in 1936.(1) Briefly, his conclusion was that the stresses in an elastic structure, under plane strain conditions, and acted upon by internal pore pressures, can be computed by first: calculating the stresses in this structure considered as impermeable

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and loaded by the appropriate pressures on the boundaries, and second: subtracting from the stresses thus obtained the actual pore pressures multiplied by the area coefficient (boundary porosity). This result, confirmed by other investigators(2) and,(3) is valid only when the pressures are continuous throughout the body of the structure and, in addition, satisfy at all points the Laplace's equation,[1] which probably would be approximately true in a uniformly porous dam with no internal drainage.

For various reasons, such as, for example, the use of richer concrete mixes near the upstream face of dams and the inclusion of drainage channels and galleries, the pressures generated within the pores of the dam material do not generally satisfy the Laplace's conditions in practice. This is amply illustrated by the measurements carried out recently on the Hiawassee dam of the T.V.A.(4) and on the foundations of several other American dams.(5)

The question which may well be posed is this: can the theory developed by Brahtz be applied with a good degree of approximation in cases where the pressure variation within the pores of the material does not follow the required law? If the answer to this were positive, the obvious corollary is that, in order to prevent tensile stresses from occurring near the upstream faces of gravity dams, it is necessary to provide there, by design calculations based on no uplift, a compressive stress at least equal to the full hydrostatic head multiplied by the area coefficient. This conclusion, arrived at by a somewhat different argument, is the basis of the well-known criterion of Maurice Levy, which, until recent days, formed the golden rule of dam design in France. This rule is still often used by many engineers, who, in order to avoid excessively heavy cross sections, assume low area coefficients—contrary to experimental evidence. A further logical consequence of acceptance of the above theory must be the fact that no amount of internal drainage is capable of reducing the worst stress conditions, a result hardly in agreement with the, often reliable, faculty of sound engineering judgment.

It was the above problem that prompted the work presented here to be undertaken. The final conclusions which will be given are not altogether unexpected and show, in fact, that the method used by many engineers in which the final stresses are assumed linear irrespective of the uplift distribution is very close to the truth. It is felt, however, that a solution based on the full elastic theory at least gives a logical background to this, rather vexing, problem.

Linear Stress Distribution

The linear distribution of vertical stresses on horizontal sections of the dam, which forms generally the basic assumption of any stress analysis, has been shown by a full elastic analysis(7) to be valid in those portions of the dam which are not very close to its foundation when uplift effects are not considered. The same result was earlier shown to be exactly true for an infinitely high triangular dam.(8) In this particular case, illustrated in Fig. 1, the seepage which obeys Laplace's equation results in pressure decreasing linearly along horizontal sections from full hydrostatic head at the upstream

---

1. Laplace's equation $\nabla^2 p = \frac{\partial^2 p}{\partial x^2} + \frac{\partial^2 p}{\partial y^2} = 0$ is normally satisfied in two dimensional seepage through a uniformly porous medium.

face to zero at the downstream face (Fig. 1a). By Brahtz's theory, which is definitely valid here, the resulting stress is given as the difference of the no uplift stress (1b) and (a) multiplied by the area coefficient. This stress is again obviously linear.

In the following work, by analogy, the stresses in a similar triangular dam will be considered but no restriction will be placed on the manner in which the pore pressures are distributed on horizontal sections. Particular attention will be given to the case illustrated in Fig. 2, where a line of drains reducing the pore pressure to zero is placed in a radial plane, the material otherwise being homogenous. Fig. 2b, showing the distribution of the stresses due to boundary loading only, and Fig. 2a, the pore pressures, give, by extending Brahtz's theory, a resulting stress distribution (2c), which is manifestly non-linear. Is this the correct stress or is it more logical to assume that, under all circumstances, the resulting stress in the solid material follows the linear distribution? It is to this question that an analysis based on the hypothetical triangular dam should give an adequate answer.

It is theoretically not difficult to extend the work done earlier by the author[7] to analyse the stresses in any dam and its foundation due to any assumed pore pressure distribution. As such work would necessitate extremely laborious computations of individual cases, it was felt that it would not be justified. In any case, one can expect reasonably that any conclusions based on an analysis of triangular dams, by analogy with the no uplift results, to be valid within the upper portions of usually shaped dams. Any deviations near the foundation level would again be of the type previously discussed.

General Theory

As is well known, the action of water pressure gradually varying through a porous body is to cause the water load to be applied to it in the form of "body forces," rather than by boundary loading.

If η is the area coefficient (known sometimes as boundary porosity), the magnitude of these body forces acting per unit volume of the material in the x and y directions respectively will be:

$$X = -\frac{\partial p}{\partial x}\eta$$

$$Y = -\frac{\partial p}{\partial y}\eta + \omega_s$$

ω_s - being the saturated unit weight of the material.

With the three stress components shown in Fig. 3, the consideration of equilibrium of the elementary volume results in the following equations:-

$$\frac{\partial \sigma_x}{\partial x} + \frac{\partial \tau_{xy}}{\partial y} - \eta\frac{\partial p}{\partial x} = 0 \qquad (1)$$

$$\frac{\partial \sigma_y}{\partial y} + \frac{\partial \tau_{xy}}{\partial x} - \eta\frac{\partial p}{\partial y} + \omega_s = 0 \qquad (2)$$

The same equations in polar coordinates using the definitions given in Fig. 4 will be:-

$$\frac{\partial \sigma_r}{\partial r} + \frac{1}{r}\frac{\partial \tau_{r\theta}}{\partial \theta} + \frac{\sigma_r - \sigma_\theta}{r} - \eta \frac{\partial p}{\partial r} + \omega_s \cos\theta = 0 \quad (1a)$$

$$\frac{1}{r}\frac{\partial \sigma_r}{\partial \theta} + \frac{\partial \tau_{r\theta}}{\partial r} + \frac{2\tau_{r\theta}}{r} - \eta \frac{1}{r}\frac{\partial p}{\partial \theta} - \omega_s \sin\theta = 0 \quad (2a)$$

Considering a two-dimensional distribution of stresses under plain strain conditions, it is necessary for the well known equation of strain compatibility to be satisfied (see "Theory of Elasticity" S. Timoshenko ref. 9).

$$\nabla^2(\sigma_x - \sigma_y) = -\frac{1}{1-\nu}\left(\frac{\partial X}{\partial x} - \frac{\partial Y}{\partial y}\right) = \frac{\eta}{1-\nu}\nabla^2 p \quad (3)$$

where $\nabla^2 \equiv \frac{\partial^2}{\partial x^2} + \frac{\partial^2}{\partial y^2} \equiv \frac{\partial^2}{\partial r^2} + \frac{1}{r}\frac{\partial}{\partial r} + \frac{1}{r^2}\frac{\partial^2}{\partial \theta^2}$

and ν is the Poisson's ratio for the material.

Introducing a stress function ϕ defining the stresses as follows, ensures that the equilibrium relationships (1), (2) or (1a), (2a) are automatically satisfied:-

$$\sigma_x = \frac{\partial^2 \phi}{\partial y^2} + p\eta - \omega_s y$$

$$\sigma_y = \frac{\partial^2 \phi}{\partial x^2} + p\eta - \omega_s y \quad (4)$$

$$\tau_{xy} = -\frac{\partial^2 \phi}{\partial x \partial y}$$

in Cartesian co-ordinates, or in polar co-ordinates:

$$\sigma_r = \frac{1}{r}\frac{\partial \phi}{\partial r} + \frac{1}{r^2}\frac{\partial^2 \phi}{\partial \theta^2} + p\eta - \omega_s r\cos\theta$$

$$\sigma_\theta = \frac{\partial^2 \phi}{\partial r^2} + p\eta - \omega_s r\cos\theta \quad (4a)$$

$$\tau_{r\theta} = \frac{1}{r^2}\frac{\partial \phi}{\partial \theta} - \frac{1}{r}\frac{\partial^2 \phi}{\partial r \partial \theta}$$

Substitution of relationships (4) or (4a) into the compatibility equation results in ϕ having to satisfy the following equation:-

$$\nabla^2(\nabla^2 \phi + kp) = 0 \quad (5)$$

where $k = \left(\frac{1-2\nu}{1-\nu}\right)\eta$

For the purposes of this paper, the Poisson's ratio of concrete will be assumed as zero, which gives $k = \eta$.

It should be noted at this stage that, if the pressure p satisfies Laplace's equation, the governing equation (5) simplifies considerably and it is from this simplification that Brahtz's conclusions are derived. However, no such assumption is mathematically required and, providing p is known as an explicit function of the co-ordinates, a solution is always possible.

Triangular Dam

In the case of a triangular dam with co-ordinates as shown in Fig. 5, it is easily seen that the distribution of stresses on similar sections must follow similar patterns. Putting this in mathematical terms, this statement is equivalent to the stresses at any point being defined as a product of two independent functions of r and θ respectively. As the pressure forces increase directly with r on the upstream face and as at all similar points the stresses must bear the same ratios to each other, it follows that any stress component can be written down in the form:-

$$r F(\theta) \tag{6}$$

where F is as yet unknown, function of θ.

Consideration of relationships (4a) implies that the stress function can be written down in the general form

$$\phi = r^3 f(\theta) \tag{7}$$

which can adequately and uniquely represent such a stress distribution.

The pressure can be quite generally defined as

$$p = r\,\psi(\theta) \tag{8}$$

Substitution of (7) and (8) into the governing equation (5), results in the following ordinary, linear, differential equation:-

$$\frac{d^4 f}{d\theta^4} + 10\frac{d^2 f}{d\theta^2} + 9f = -k(\psi + \psi'') \tag{9}$$

The right hand side of this equation being known in terms of θ for a given pressure distribution in the pores of the material, a solution for $f(\theta)$ is easily obtained. If ψ and ψ'' are both continuous, the general solution is of the form:-

$$f(\theta) = A\cos\theta + B\sin\theta + C\cos 3\theta + D\sin 3\theta + \text{Particular integr.} \tag{10}$$

To evaluate the constants occurring in the solution, it is necessary to consider the conditions at the boundaries. These clearly are:-

$$\begin{aligned} &\text{when } \theta = 0 && \sigma_\theta = -(1-\eta)p \text{ or } 6f(0) - \omega_s + \psi(0) = 0 \\ & && \tau_{r\theta} = 0 \qquad f'(0) = 0 \\ &\theta = \alpha && \sigma_\theta = 0 \qquad \text{or } 6f(\alpha) - \omega_s \cos\alpha + \eta\psi(\alpha) = 0 \\ & && \tau_{r\theta} = 0 \qquad f'(\alpha) = 0 \end{aligned} \tag{11}$$

Solution for a Particular Pressure Distribution

A case of special interest is that already mentioned and illustrated in Fig. 2. In this, a line of drains at which the pressure reduces to zero is placed at the angle β . It is assumed that although the region between the drains and the downstream face is at zero pressure, the material there is saturated. This is done to eliminate the difficulty of having varying unit weights of the material and would not, in any practical case, introduce a significant error if this was not true. This distribution of pressure assumed is often approximated to in practice and in practically all measured cases is a closer assumption than the linear variation from upstream to downstream faces.

In the case illustrated, the pressure decreases from its full upstream value of wy, linearly with x, to zero at the line of drains. This, written in polar co-ordinates, is:-

$$\begin{aligned} &0 \leqslant \theta \leqslant \beta && \psi = \frac{\omega \sin(\beta - \theta)}{\sin\beta} \\ &\beta < \theta \leqslant \alpha && \psi = 0 \end{aligned} \tag{12}$$

where ω = unit weight of water.

These particular values of ψ give a zero value of the right hand side of equation (9) at all points with the exception of the line $\theta = \beta$ where clearly ψ " becomes infinite. This introduces a discontinuity into the equation and more elaborate mathematical methods have to be used to obtain the solution. An outline of the required calculations is presented in Appendix I.

The most important result of these calculations is concerned with the distribution of vertical stresses on horizontal planes. It is found that this distribution is of the form given by two straight lines of different slope in each of the two regions. The values of stress defining these lines are given by the following expressions:-

$$\theta = 0 \qquad \sigma_y = \frac{\omega y}{\sin^2\alpha}\left\{1 - \sin^2\alpha\,(1+\gamma-\eta) - \eta\sin^2(\alpha-\beta)\right\}$$

$$\theta = \beta \qquad \sigma_y = \omega y\left\{\eta\frac{2\sin^2(\alpha-\beta)}{\sin^3\alpha}\cos\alpha\tan\beta - \frac{\gamma\sin(\alpha-\beta)}{\sin\alpha\cos\beta} + \right. \tag{13}$$
$$\left. + \cot^2\alpha\,(1 - 2\cot\alpha\tan\beta)\right\}$$

$$\theta = \alpha \qquad \sigma_y = -\frac{\omega y}{\sin^2\alpha}\left\{\eta\sin(\alpha-\beta)\sin\beta\cos\alpha + \cos^2\alpha\right\}$$

In these $\gamma = \frac{\omega_s}{\omega}$

Similar expressions are derived for the shear stress, which again follows a straight line distribution within each region.

$$\theta = 0 \qquad \tau_{xy} = 0$$

$$\theta = \beta \qquad \tau_{xy} = \omega y\left\{\eta\frac{\sin^2(\alpha-\beta)}{\sin^2\alpha} - \cot^2\alpha\right\}\tan\beta \tag{14}$$

$$\theta = \alpha \qquad \tau_{xy} = -\omega y\left\{\eta\frac{\sin(\alpha-\beta)\sin\beta}{\sin\alpha} + \cot\alpha\right\}$$

Quantitative Results

In Tables I and II below are shown the values of vertical stresses calculated by means of expressions (13) for two different values of angle α, and varying angle β. In both calculations the area coefficient of unity is taken and the specific gravity of the saturated material is assumed as 2.420. For comparison in each case the stress resulting from an assumption of linear stress distribution is given (in bracketed figures).

Somewhat better visualisation of the results is obtained from inspection of Figs. 6 and 7. In these, both the vertical and shear stress distributions have been shown for two different area coefficients. For comparison purposes in each case curves (b) giving the stress distribution by extension of Brahtz's theory and (c) those which assume a linear distribution of vertical stress in the material are included.

The most important observations here will concern the distribution of vertical stresses. It is immediately observed that the theoretically correct distribution lies in all cases between that resulting from the assumptions (b) and (c) of the preceding paragraph. This indeed is a result which would have been logically anticipated. Of great practical importance, however, is the fact that in all cases the departure of actual stresses from linear distribution across a horizontal section is rather small. In Fig. 8, the error resulting from the linear assumption in one of the cases analysed is plotted as a function of the angle defining the position of the drainage plane. As expected, whenever this plane coincides with one of the faces of the dam, this error is zero, all approaches resulting in linear solutions. Near the upstream face,

TABLE I

$\eta = 1$; $\gamma = 2.420$; $\alpha = 40^\circ$ (All stresses compressive)

β	$\theta = 0$ $-\frac{\sigma_y}{\omega y}$		$\theta = \beta$ $-\frac{\sigma_y}{\omega y}$		$\theta = \alpha$ $-\frac{\sigma_y}{\omega y}$	
0	(1.000)	1.000	(1.000)	1.000	(1.420)	1.420
5	(0.802)	0.796	(0.876)	0.878	(1.514)	1.513
10	(0.624)	0.605	(0.826)	0.834	(1.586)	1.581
15	(0.463)	0.432	(0.838)	0.858	(1.638)	1.622
20	(0.321)	0.283	(0.904)	0.937	(1.666)	1.637
25	(0.197)	0.162	(1.014)	1.053	(1.667)	1.623
30	(0.097)	0.073	(1.155)	1.189	(1.635)	1.581
35	(0.027)	0.018	(1.305)	1.320	(1.558)	1.513
40	(0.000)	0.000	(1.420)	1.420	(1.420)	1.420

TABLE II

$\eta = 1$; $\gamma = 2.420$; $\alpha = 32^\circ\ 44'$ (Tensile stresses shown negative)

β	$\theta = 0$ $-\frac{\sigma_y}{\omega y}$		$\theta = \beta$ $-\frac{\sigma_y}{\omega y}$		$\theta = \alpha$ $-\frac{\sigma_y}{\omega y}$	
0	(0.000)	0.000	(0.000)	0.000	(2.420)	2.420
5	(-0.254)	-0.259	(0.126)	0.128	(2.538)	2.537
10	(-0.473)	-0.489	(0.375)	0.384	(2.619)	2.614
15	(-0.660)	-0.682	(0.725)	0.744	(2.664)	2.647
20	(-0.812)	-0.834	(1.157)	1.182	(2.666)	2.637
25	(-0.924)	-0.938	(1.646)	1.666	(2.619)	2.584
30	(-0.990)	-0.992	(2.155)	2.160	(2.512)	2.489
32.73	(-1.000)	-1.000	(2.420)	2.420	(2.420)	2.420

the maximum error reaches 3.8% and at the downstream face 5.4% of the full hydrostatic pressure. In both places, the linear assumptions tend to result in higher compressive stresses, thus being not on the side of safety. A consideration, however, of the magnitude of error involved for dams of usual heights shows, in fact, that it is entirely insignificant.

It should be noted that any extension of Brahtz's theory beyond its limits of strict applicability results in entirely misleading conclusions. Effective drainage within the body of a dam, by reducing the total uplift force, results in higher compressive stresses at the upstream face, irrespective of the fact that in that region full hydrostatic head may be active within the pores.

Although the above remarks are based on the results of a particular type of pore pressure distribution, numerical computation of other cases confirms their generality. In Appendix II, formulae are given for a case when the pressure at the drainage line falls to any given fraction of the full hydrostatic head. These and others where, for example, parabolic distributions of pressures have been assumed, all show, in fact, departures from linearity smaller than in the examples quoted.

The distribution of shear stresses is of some interest. Figs. 6 and 7 show that here the departure from linearity is very much more pronounced. A superficial consideration of the principles of statics shows that this indeed must be the case. For a constant downstream slope, the shear stress at the face bears a fixed ratio to the vertical stress, which as is clearly seen varies with the position of the drainage. On the other hand in all cases the integrated force of the shear stresses must be equal to the horizontal thrust of the water, which is quite independent of the particular pressure distribution. As in the particular case of no internal drainage, the linear distribution of shear stresses is achieved it must follow that in all other instances departures from it will be observed. It is but seldom that the engineer is interested in the exact shear stress distribution and all that requires noting is the fact that once the vertical stresses are computed, it only requires rather simple static calculations to derive the answer. S. Leliavsky[11] presents an excellent method of shear stress calculations which may, with ease, be adapted for any circumstances required.

CONCLUSIONS

It is hoped that the main result, which appears at the end of the somewhat elaborate calculations, will by its simplicity appeal to the designer and help to provide in some cases a more rational approach to the problem so often encountered. The use of such simplifying assumptions as for example a linear variation from 50% of hydrostatic head at the upstream face to zero at the downstream face, obviously at variance with the truth, should be simply interpreted as giving statically equivalent distributions and not as representing the facts.

The author would like to express his thanks to Mr. J. Park for assistance in checking the calculations, carrying out numerous computations and contributing the solution given in Appendix II.

APPENDIX I

With pore pressures distributed according to relationship (12), the governing equation (9) takes on the following form:-

$$\frac{d^4 f}{d\theta^4} + 10\frac{d^2 f}{d\theta^2} + 9f = -K\,\delta(\theta-\beta) \qquad (9a)$$

δ being the Dirac's impulse function such that

when $\theta \neq \beta$ $\qquad \delta(\theta-\beta) = 0$

$\theta = \beta$ $\qquad \delta(\theta-\beta) = \infty$

and $$\int_{-\infty}^{+\infty} \delta(\theta-\beta)\,d\theta = 1$$

The value of the multiplying constant K can be obtained by integration of the right hand side of equation (9).

$$K = \int_{\theta<\beta}^{\theta>\beta} k(\psi+\psi'')\,d\theta$$

and results in

$$K = \frac{k\omega}{\sin\beta}$$

or if $\nu = 0$ $\qquad K = \eta\frac{\omega}{\sin\beta}$

Multiplying both sides of equation (9a) by $e^{-p\theta}$ and integrating between zero and infinity, gives by the usual methods of Laplace's transformation (see pages 1 - 10, ref. 10), the following:-

$$(p^4+10p^2+9)\,\bar{f} = p^3 f(0) + p^2 f'(0) + p[f''(0)+10f(0)] + f'''(0) + 10f'(0) - Ke^{-p\beta}$$

where $\bar{f} = \int_0^\infty f e^{-p\theta}\,d\theta$ (Laplace transform of function f) and f(0), f'(0) etc. stand for the values of the function f and its derivatives when $\theta = 0$.

The boundary conditions at $\theta = 0$ (11) give:-

$$f(0) = \frac{\omega_s - \omega}{6} \quad ; \quad f'(0) = 0$$

Substituting the above into the transformed equation, solving for $\bar{f}$ in terms of partial fractions and using standard forms of inverse transformation, it is found that

$$f(\theta) = -\frac{1}{8}\left(\frac{\omega_s-\omega}{6} + f''(0)\right)\cos 3\theta - \frac{f'''(0)}{24}\sin 3\theta +$$
$$+ \frac{1}{8}\left\{\frac{3}{2}(\omega_s-\omega) + f''(0)\right\}\cos\theta + \frac{f'''(0)}{8}\sin\theta +$$
$$+ K\,H(\theta-\beta)\left\{\frac{1}{24}\sin 3(\theta-\beta) - \frac{1}{8}\sin(\theta-\beta)\right\}$$

in this $H(\theta - \beta) = 0$ when $\theta < \beta$
and $H(\theta - \beta) = 1$ when $\theta \geq \beta$

The unknown values of f''(0) and f'''(0) can now be obtained from the now known relationship for f and the boundary conditions at $\theta = \alpha$, which are

$$f(\alpha) = \frac{\omega_s \cos\alpha}{6}$$
$$f'(\alpha) = 0$$

This results in

$$f''(0) = -\eta\omega\,\frac{\sin^2(\alpha-\beta)}{\sin^2\alpha} - \frac{(\omega_s-\omega)(1+\cos^2\alpha)}{2\sin^2\alpha} + \omega_s\cot^2\alpha$$

$$f'''(0) = \eta\omega\,\frac{\sin^2(\alpha-\beta)}{\sin^3\alpha}(2\cos\alpha + \sin\alpha\cot\beta) + (\omega_s-\omega)2\cot^2\alpha +$$
$$+ \frac{\omega_s\cos\alpha}{\sin^3\alpha}(1-3\cos^2\alpha)$$

The expression for f is now explicitly known in terms of the data provided and hence the stress function ϕ can be expressed in terms of r and θ. An easy substitution will convert this into a function of x and y, which is more convenient for evaluation of stresses in the Cartesian system. The results obtained for the vertical and shear stresses give linear relationships with x in each of the regions $0 < \theta < \beta$ and $\beta < \theta < \alpha$ and the expressions for the key values are quoted in the main text (13 and 14).

APPENDIX II

By methods analogous to those given in Appendix I, Mr. J. Park has derived the stress distribution for the case where the pore pressure decreases

linearly with x from its full hydrostatic value to an n-th part of it at the line of drains and thence again linearly to zero at the downstream face. The stresses are distributed again linearly within each region with the following key values:-

$$\theta = 0 \qquad \sigma_y = -\frac{\omega y}{\sin^2\alpha}\left\{\left(\delta \sin^2\alpha - \eta \sin^2\alpha - \cos^2\alpha\right) + \right.$$
$$\left. + \eta \sin(\alpha-\beta)\left[\sin(\alpha-\beta) - n\cos\beta\sin\alpha\right]\right\}$$

$$\theta = \beta \qquad \sigma_y = -\frac{\omega y}{\sin^2\alpha}\left\{\delta\frac{\sin\alpha\sin(\alpha-\beta)}{\cos\beta} - \cos^2\alpha\left(1-\frac{2\tan\beta}{\tan\alpha}\right) - \right.$$
$$- \eta n \sin^2\alpha - 2\eta\sin(\alpha-\beta)\frac{\tan\beta}{\tan\alpha}\left[\sin(\alpha-\beta) - \right. \qquad (15)$$
$$\left.\left. - n\cos\alpha\sin\beta\right]\right\}$$

$$\theta = \alpha \qquad \sigma_y = -\frac{\omega y}{\sin^2\alpha}\left\{\cos^2\alpha + \eta\sin(\alpha-\beta)\cos\alpha\sin\beta - \right.$$
$$\left. - \eta n \sin\alpha\cos\alpha\sin\beta\cos\beta\right\}$$

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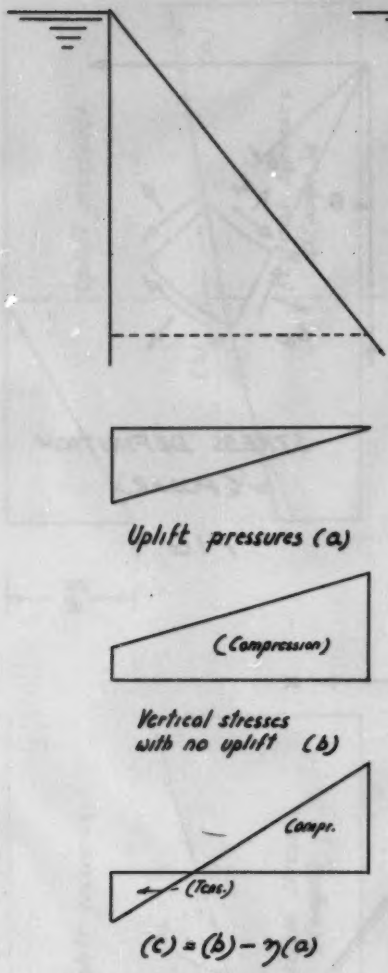
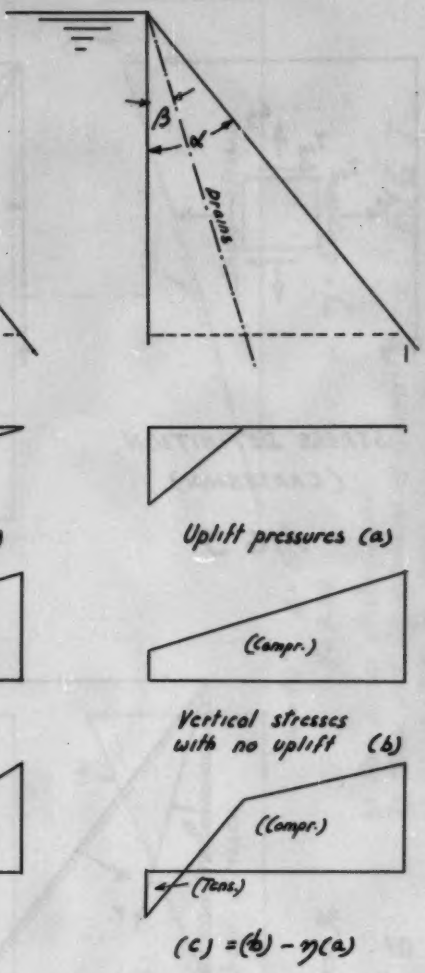


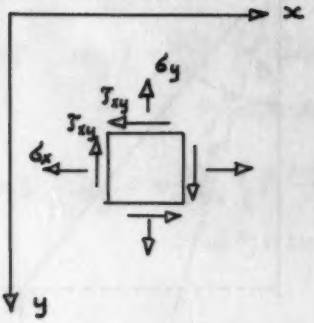
RESULTANT STRESSES WITH LINEAR UPLIFT DISTRIBUTION

FIG 1



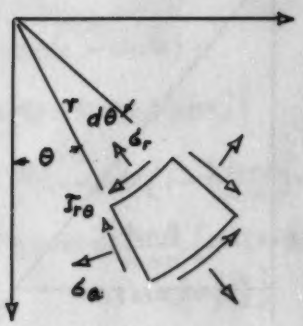
EXTENSION OF BRAHTZ'S THEORY TO THE CASE OF NON-LINEAR UPLIFT DIST.

FIG. 2



STRESS DEFINITION
(CARTESIAN)

FIG 3



STRESS DEFINITION
(POLAR)

FIG 4

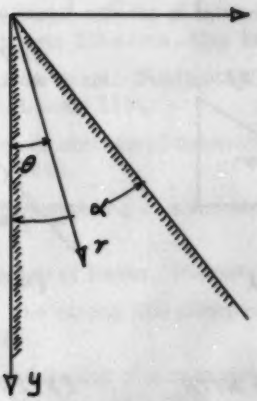
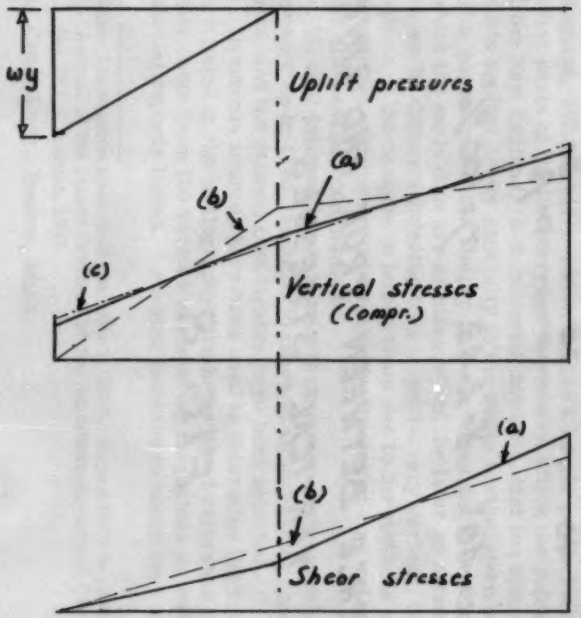
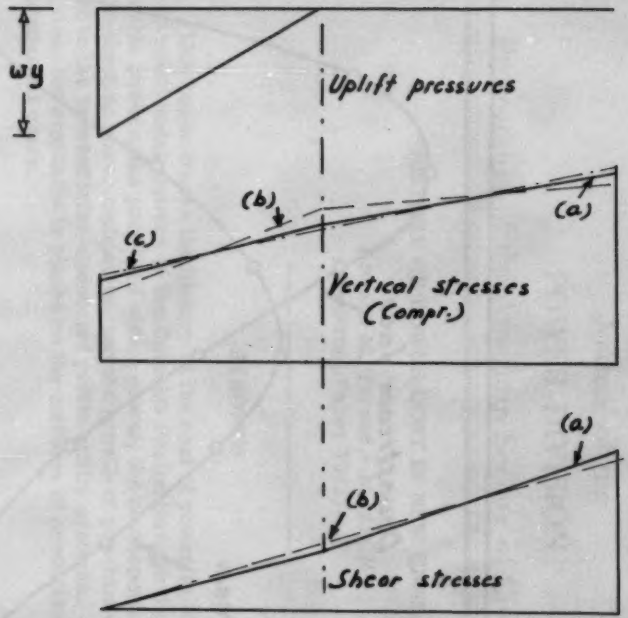


FIG 5





$\alpha = 40°$ $\beta = 20°$

$\gamma = 2.42$

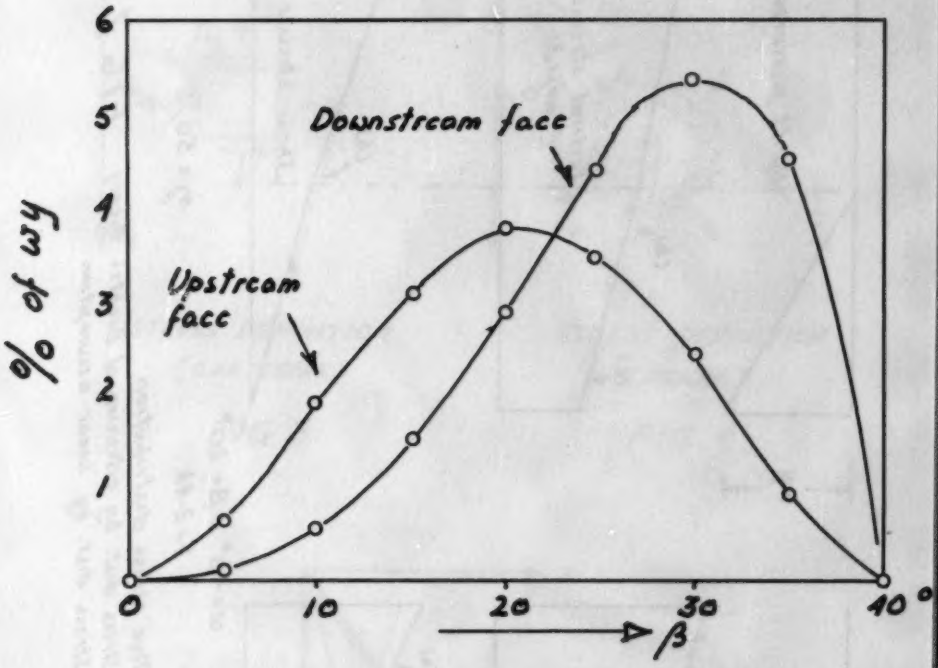
$\eta = 100\%$

FIG 6

$\eta = 50\%$

FIG 7

a) True stress distribution
b) Stress dist. by extension of Brahtz theory
c) Stress dist. by linear assumption



$\alpha = 40°$ $\gamma = 2.42$ $\eta = 100\%$

DIFFERENCE BETWEEN TRUE AND LINEAR VERTICAL STRESSES

FIG 8

Paper 1043 PO 4

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

HISTORY OF POWER COST IN NEW ENGLAND

Howard M. Turner,[1] M. ASCE
(Proc. Paper 1043)

SYNOPSIS

This paper traces the history of the cost of power in New England from the small water powers of the Colonial Period through the large industrial water powers, the growth of steam power, the development of the early hydroelectric power companies, and the growth of purchased power by industry, up to the present inter-connected public utility systems. Cost figures are given, but emphasis is placed on the methods of production rather than on statistical costs.

The title of this paper refers to the power costs in New England. New England, with its early industries has a longer power history than most of the other parts of the country, and the data on which the paper is based come from New England. It is believed, however, that the history of its power costs applies pretty generally to the whole northeastern part of the country. This paper traces the history of power costs not so much by statistics, for which the early data are so meagre and the later so extensive that the results shown by different investigators might be very various, but more from the causes of the changes in power costs due to the methods and efficiency of production. These are illustrated by figures of cost selected to be typical for the different periods studied.

No real information is available on power costs up to about 1820 and very meagre up to 1840. Since 1910 there is an increasing amount of data available, and the general cost figures have been selected.

Numerous sources have been used to determine the historic data. These are listed in the bibliography attached without reference in the text. Many data came from the various engineering magazines as well as from books and special reports listed. I am also indebted to assistance from men in the

Note: Discussion open until January 1, 1957. Paper 1043 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 82, No. PO 4, August, 1956.

1. Cons. Engr., Boston, Mass.

following companies: Proprietors of the Locks and Canals, Lowell, Mass.; Holyoke Water Power Co.; Central Maine Power Co.; Public Service Company of New Hampshire; New England Power Co.; Boston Edison Co.; Western Mass. Electric Co.

The history of power divides itself into several periods: first the Colonial period, which may be taken from 1650 to about 1812, when small industry dating from early Colonial times was beginning to get started. An early example is shown in the recent restoration of the Saugus Iron Works in Saugus, Massachusetts, built in 1650. Here there are seven water wheels, mostly of the overshot type. One 15 ft. 10 in. in diameter develops 8.4 HP with an efficiency of 70%. The wheels drive bellows, a trip hammer, rolling and slitting mills.

A list of manufacturers on the Charles and Neponset Rivers, small rivers near Boston, shows the total industries on those rivers to be as follows: in 1650, three; in 1700, eight; in 1750, eighteen; in 1812, forty-six. In the early years these were practically all saw, grist, and fulling mills, but at the end of the period there were a total of eighteen different industries, including paper, cotton, and iron working mills. These were all small mills operated by water power. These were mostly industries in the present sense of the word, producing goods for a wider distribution than just for the local demand.

During this period there were a great many local mills scattered through New England driven by water power, though there is one record of a woolen mill driven by a 30 HP steam engine in 1810. One finds many small streams with the remains of a series of small water power plants. The dams in many cases were built mostly of stone masonry. The equipment was probably small undershot wheels. Wooden vertical tub wheels were used as early as 1800. It took about 4 HP to turn the stones of a grist mill, and of course it was not necessary to operate continuously, so that small streams gave sufficient water to operate the plants when the power had to be used.

It is difficult to find any power cost figures. What cost figures there are are not divided. The figures of $10,000 and $15,000 given for a mill include the total investment.

The development of the Boston Manufacturing Company plant on the Charles River in Waltham, 1815, was the first complete cotton manufacturing plant and began a new era in the use of industrial power. The water power had a head of 13 feet, the river a drainage area of 248 sq. mi. The mill was a 4-storey building, 90 ft. by 45 ft. The use of industrial power increased as industry developed, culminating in the large water power developments of the 1840's, developed by the canal companies typified by those at Lowell, Lawrence, Holyoke, Massachusetts, and Lewiston, Maine. These developments may be said to be the start of the modern power business in this country, and here we get the first basis from which we can obtain power costs with considerable accuracy.

These canal companies were all somewhat similar. They existed in some other parts of the country also. To divide up the large amount of power developed by a large river the water was distributed by canals. Fig. 1 shows the canal plan at Lowell. A certain quantity of water at a given head was sold to each customer, in some cases stockholders in the enterprise, located along the canals. The total head was often divided into one or more levels to increase the distribution of the power. The mill bought potential power measured by "mill powers," defined as so many cubic feet per second under a given head. This was a permanent right which went with the land. A "mill

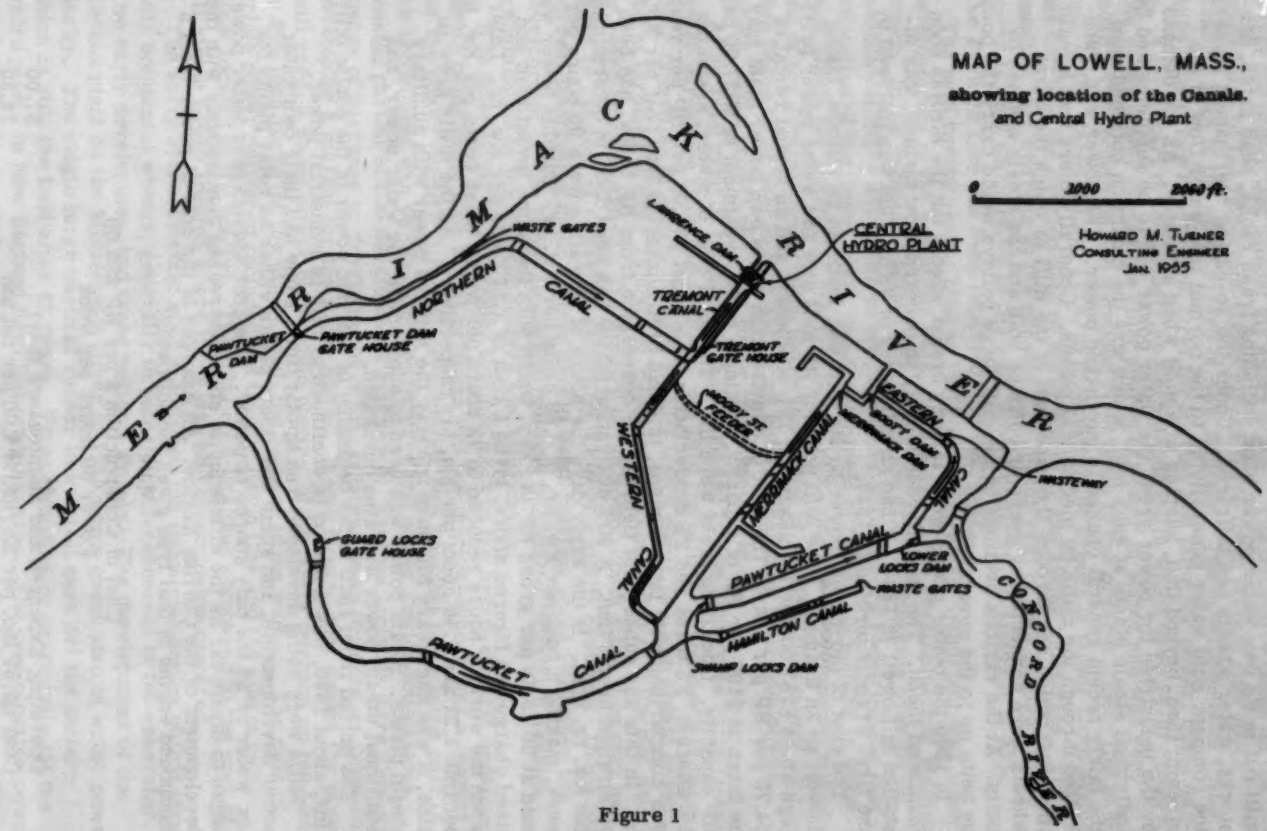

MAP OF LOWELL, MASS.,
showing location of the Canals.
and Central Hydro Plant
0 1000 2000 ft.
HOWARD M. TURNER
CONSULTING ENGINEER
JAN. 1935
MERRIMACK RIVER
CENTRAL HYDRO PLANT
WASTE GATES
LAWRENCE DAM
NORTHERN CANAL
TREMONT CANAL
TREMONT GATE HOUSE
PAWTUCKET DAM
PAWTUCKET DAM GATE HOUSE
MOODY ST. FEEDER
EASTERN CANAL
MIDDLESEX CANAL
WESTERN CANAL
GUARD LOCKS GATE HOUSE
PAWTUCKET CANAL
LOWER LOCKS DAM
WASTE GATES
HAMILTON CANAL
SWAMP LOCKS DAM
CONCORD RIVER

Figure 1

power" in Lowell and Lawrence was about 85 theoretical horsepower. The mill put in its own water power plant including trash racks, head gates, penstocks, water wheels and tailraces. Under the so-called "permanent leasees" the company sold water generally up to the normal low-water flow of the stream; then, beyond that, surplus was sold when available. As the demand by the industries for power grew, the water power had to be supplemented by individual steam plants at each mill, which was the condition obtained until the fairly recent general change to purchased power. The power was transmitted through the mills by mechanical means. Generally there was a main line shaft which ran through the basement of the mill to which the water wheels were connected by gears or belts or both and the steam engines by belts. At the various sections of the mill large pulleys would carry the power by belts up to the several floors.

In 1840 the power plants were largely breast wheels but the turbine soon displaced these. The first Boyden Wheel, an outward-flow turbine, was installed in Lowell in 1844. It had an efficiency of 78%. This was followed by the first Francis turbine in 1847.

The cost of permanent power to the mill was made up as follows, taking Lowell as an example. The mill paid for its mill site and water power privilege about $157.00 per HP assuming 75% efficiency, and an annual rental of $4.70 per HP per year. The cost of construction of its own power plant may be taken at $100 per HP of capacity, making a total investment of $257.00 per HP capacity. Operating costs were low. The total cost of power, assuming 6% interest, 3% taxes and 2% depreciation, was about $31.00 per HP per year. For comparison with later figures, this has been converted into cost per Kw. hr. In this and subsequent figures, the conversion from HP to Kw. has been by straight proportion. A 90% utilization factor has been used in determining the Kw. hr. cost. In the early days the hours of operation were 90 hours per week, making a cost per Kw. hr. of 0.98 cents per Kw. hr. Surplus power was sold at 59 cents per gross HP per day, which with the same investment and fixed charges as figured above, gives a cost of 1.17 cents per Kw. hr. Later when the working hours reduced to 72 hours per week, these costs per Kw. hr. became 1.22 and 1.46 cents per Kw. hr. respectively. As the mills grew in size, it was necessary for them to relay the surplus power with auxiliary steam plants. Steam power was used to supplement the water power, and additional steam power was also developed.

The period from 1860 to the beginning of electric power of the public utilities, about 1890, was a period of continued growth of industrial power.

Mills were being developed during this time; some had their own water power developments, and some had steam plants either as relay plants for low water or as the only source of power. The cost advantage of water power began to disappear as the efficiency of steam increased continuously with the development of the Corliss Engine. An example of the cost of power from a compound engine of that type gives the following:

Cost of Construction $ per HP $55.00
Power 60 hours per week

Operating Expenses and Maintenance (say)	$7.00
Coal @2.55 lbs. per HP/hr. @ 4.00 per net ton	14.30
Fixed Charges @ 12%	6.60
Cost per HP/year	$27.90

Equivalent to 1.35 cents per Kw. hr.
With coal at $3.00 the cost would be 1.10 cents

The power situation in Maine was different from the rest of New England in that the power was almost all water power. The extensive lakes on the headwaters of the important Maine rivers made their stream flow much steadier than in the other rivers of New England. The development of these lakes as storage reservoirs, first for logging and later for power, came early. A large part of the industries were paper mills which used 24-hr. power. The water power stayed the chief source of power in Maine and was very cheap. This condition has extended right down to the present time. The passage of the Fernald Law, 1915, in prohibiting the export of power from Maine has confined the development of water power to the Maine industries. The result is that whereas at the present time in the rest of New England water power is only about 16.5% of the total power capacity, in Maine it is 55%. The Fernald Law has been repealed this last year.

The next twenty years, 1890 to 1910, were extremely important with many developments in the industrial field, chiefly due to the development of electric power. In the industrial plants this meant the gradual changing over from mechanical drive with belts to electric drive with motors requiring the electrification of the power plant, the development of the steam turbine in industrial plants for generating electric power replacing the old compound engines, and of the greatest importance the development of public utility power which then began to be sold to manufacturing plants displacing their own power plants. This growth of utility power is shown on Fig. 2.

After 1910 most of the growth in power used by the industries came from power purchased from the utilities. By 1910 the utility companies had generating capacities of nearly 500,000 Kw., of which about 70% was steam power. Steam turbines of over 15,000 Kw. capacity with coal rates of 2 lbs per Kw. hr. were being installed. The history of the power costs since then may be taken as the cost of power developed by the public utility systems. There are several interesting dates on this development starting about 1910 when the development of the large modern hydro-electric station with high-tension transmission to the market centers was undertaken.

It is interesting to remember that for most of the hydro-electric developments being made then a real problem was the market—where was a market for the power output? Gradual development was possible but a large part of the investment was on the dams, waterways and power station foundations, making partial initial development expensive. Various methods were used by means of purchase or development of steam power later to serve as relay for developing a market for the water power. It was figured that except for paper mill uses, a water power plant could dispose of its power perhaps on the 40% to 50% load factor of the utility load, the balance being wasted. The cost of prime power delivered included the transmission line and the steam relay plant located at the market. The total cost was thus compared to the cost of all steam power located where there was no transmission required.

The early hydro-electric station followed the industrial design of that time with horizontal wheels, generally in pairs direct-connected to the generator. One early development had three vertical wheels. The development of the Keokuk plant on the Mississippi River in 1913 was a great step in water power design. The single large vertical runner in a scroll case set the pattern which is still the best after 42 years. From that time on practically all the hydro plants in New England have followed this plan. The cost of water power development has varied as always with the qualities of the site. Some developments made in the period 1910 to 1914 when prices rose due to World

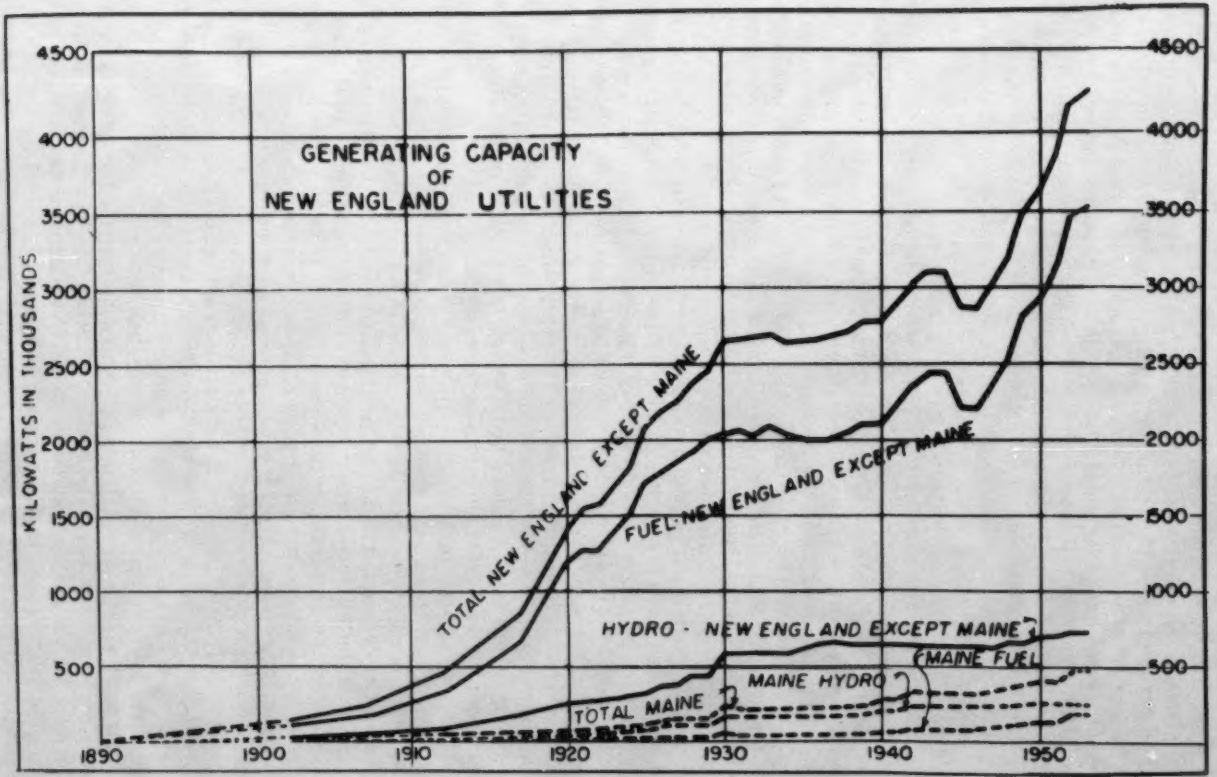

GENERATING CAPACITY
OF
NEW ENGLAND UTILITIES
KILOWATTS IN THOUSANDS
4500
4000
3500
3000
2500
2000
1500
1000
500
1890
1900
1910
1920
1930
1940
1950
TOTAL NEW ENGLAND EXCEPT MAINE
FUEL-NEW ENGLAND EXCEPT MAINE
HYDRO - NEW ENGLAND EXCEPT MAINE
MAINE FUEL
MAINE HYDRO
TOTAL MAINE

Figure 2

War I cost about $115 up to $248 per HP, (the latter with very extreme costs for water rights). Hydro power output, both prime and secondary, was given an average value in one case about this time of 0.8 cents per Kw. hr. at the switchboard. The great question of water power vs. steam power was a matter of great speculation. The idea of combined operation with increased efficiency came later. One was a water power man or a steam man. In the last analysis, however, of course, the cost of steam power was then as now the ultimate yardstick of the value of water power, except in Maine where very little steam power was in use.

About 1911 large vertical steam turbines began to be installed (smaller ones had been in use for some years) and the horizontal type became standard after about 1915. Steam plant costs in 1910 were less than $100/Kw. At this time smaller public utility companies were being organized into larger systems and generating facilities were being consolidated and interconnected with the larger companies. Industrial power distribution had been pretty well electrified; for example by 1910 all the industrial water power plants in Lowell were hydroelectric.

Then in 1915 the first world war brought a great increase in prices. In the three years 1915-1919 all costs increased about 100%. The effect on the cost of power was tempered in the case of steam plants by use of large turbine units at a more economical cost per Kw. for buildings and equipment. The cost of fuel showed a corresponding price rise which led to the use of the coal clauses in power rates increasing the price per Kw. hr. by the price paid for coal.

The year 1925 is taken as the beginning of the present era. The load kept on increasing. At that time interconnection of utilities systems began; the "superpower" idea had been under discussion for a few years. This led to the more economical use of steam power by improved plant factor operation. It also permitted the development of hydro plants submerged in the total load. This operation made possible the use of water power to its full entent. The maximum capacity value could be obtained by operating the hydro plant on the short-hour load on top of the load curve during low water and thus spreading the available Kw. hrs. over as short a time as possible, giving the maximum Kw. capacity. The maximum output could be obtained by using the hydro plant during high water at the base of the load curve so that all the available Kw. hrs. could be obtained during long hours. Fig. 3 shows this method of operation of the combined steam and hydro load of the Connecticut Power Exchange.

Fig. 4 shows the gain that this gives in hydro production. An assumed run of the river plant operated on a load of a total peak of four times the hydro capacity would give a prime capacity value of 85% higher and produce over 20% more Kw. hrs. than if operated on a similar load with a peak the same capacity as the plant.

At about this time the method of charging for power and also of computing power costs and values underwent a change. The early units in the manufacturing plants was generally the horsepower year. This, of course, would depend on the load factor. It would be different for the same plant operating a paper mill running long hours and a textile mill running 60 hours a week. The public utility power in the early days was estimated and sold by cents per Kw. hr., a unit that suffered from the same trouble as the horsepower year, in that the cost per kw. hr. changed with the load factor. Then came the separation of the demand or capacity measured in kilowatts and the

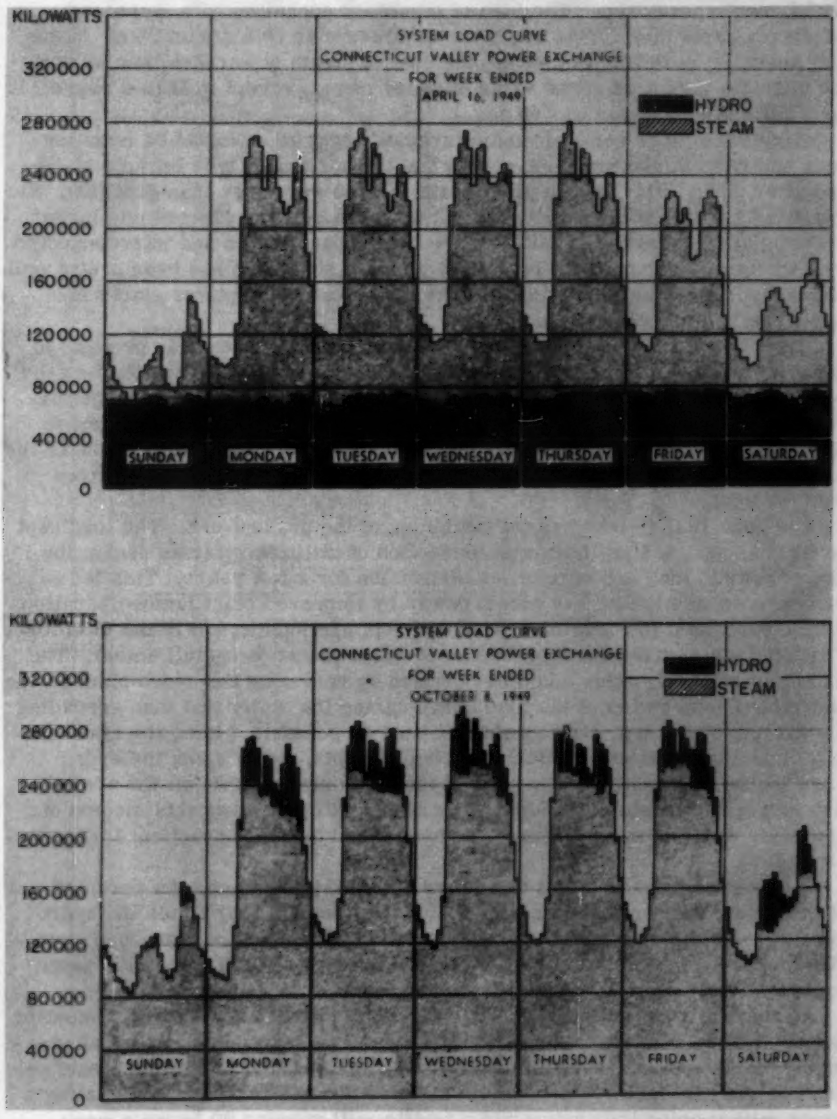
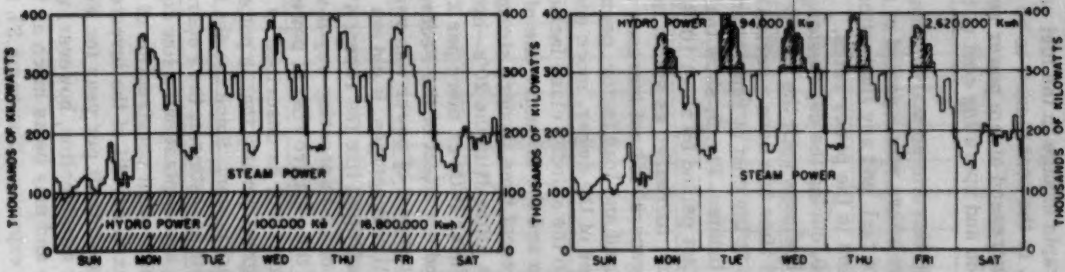

KILOWATTS
SYSTEM LOAD CURVE
CONNECTICUT VALLEY POWER EXCHANGE
FOR WEEK ENDED
APRIL 16, 1949
HYDRO
STEAM
320000
280000
240000
200000
160000
120000
80000
40000
0
SUNDAY
MONDAY
TUESDAY
WEDNESDAY
THURSDAY
FRIDAY
SATURDAY
KILOWATTS
SYSTEM LOAD CURVE
CONNECTICUT VALLEY POWER EXCHANGE
FOR WEEK ENDED
OCTOBER 8, 1949
HYDRO
STEAM
320000
280000
240000
200000
160000
120000
80000
40000
0
SUNDAY
MONDAY
TUESDAY
WEDNESDAY
THURSDAY
FRIDAY
SATURDAY

Figure 3

HIGH WATER WEEK LOW WATER WEEK

100,000 Kw LOAD
PRIME CAPACITY 51,000 Kw
ANNUAL ENERGY 367,500,000 Kwh



HIGH WATER WEEK LOW WATER WEEK

400,000 Kw LOAD
PRIME CAPACITY 94,000 Kw
ANNUAL ENERGY 465,500,000 Kwh

OPERATION OF 100,000 Kw HYDRO POWER USED ON PUBLIC UTILITY LOADS OF DIFFERENT SIZES

Figure 4

energy or output measured in Kw. hrs. This first appeared in the rates for selling power. The customer paid a demand charge plus an energy charge. Then it was applied to all power cost figuring. Insofar as it is possible that more exact cost figures permit cheaper rates, this undoubtedly contributed to the keeping down of the cost of power sold.

At this time the rapid change in the fuel economy had tapered off, reaching 1 lb. per Kw. hr. in the best plants, to be slowly reduced by about 25% in the next 30 years. Increased steam power economy was obtained by the decrease in construction and operating cost of the steam plants due to larger Kw. capacities.

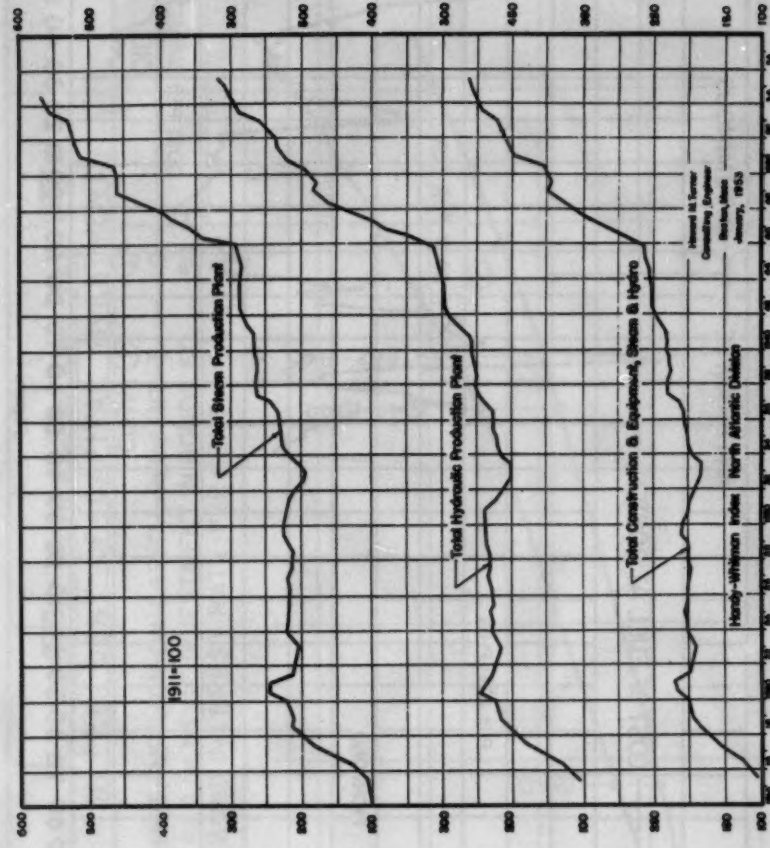
From 1925 to 1930 there is no marked change which affected the general power cost. This five years was a period of steady growth and stable prices of construction and of fuel, and any fluctuations in the cost of coal were taken care of by coal clauses in the power rates.

From 1930 to 1940, during the depression, there was a very small increase in the power business until the end of World War II in 1945. From then until the present time has been a period of increased costs of construction giving increases in the cost of generation of power and of power rates. The Handy Whitman Indices, Fig. 5, show that the cost of constructing a steam plant rose in that period by about 100%—hydro stations by nearly as much. The cost of fuel increased as shown in Fig. 6 by over 100%. However, the cost of power in general did not increase as rapidly as these indices. The increase in the load and load factor, permitting more economical operation, the increased use of the newer, more economical stations as the early plants were retired to the standby class increased the general efficiency of steam plant operation and the intensive use of hydro stations was, if anything, increased. At the present time modern steam plants cost over $200 a kilowatt instead of $100 a kilowatt in the 20's—hydro stations correspondingly higher, from $300 to over $400 or more per kilowatt to develop.

Later in this period the effect of the Federal income tax began to be felt. The effect of this on the cost of power was very marked. As long as the income tax was in small percentages, it did not become a great factor. When, however, it reached anything like the recent 47%-52%, it entered into the cost of all power and in a more interesting way it enters in unevenly. The effect of this income tax on the relative cost of power between steam plants and hydro plants is very marked, as described by Maurice R. Scharff.[2] This income tax presents a great penalty that is put on the development of water power compared to fuel power. Since the net income of a public utility company is a function of its investment, the Federal income tax is a fixed charge on the investment. A steam plant has a low construction cost compared to a hydro plant. With steam plant construction of about $200 per kilowatt of rated capacity an income tax of say 3% of the invested capital (or cost of construction), means $6.00 per kilowatt per year for Federal income tax on the steam station. Water power construction, however, now often costs $400 per kilowatt of rated capacity and may be as much as $500 or more per kilowatt of the prime dependable capacity. The same 3% income tax applied to the water power station means a cost of $12 to $15 per kilowatt per year of dependable capacity compared to the steam plant's $6.

The curves on Fig. 7 show the percentage of investment that the income

2. Maurice R. Scharff—Valuation and Depreciation Related to Income Tax, Transactions 117, 1952, p. 341.



COST TREND OF GENERATING PLANTS Figure 5

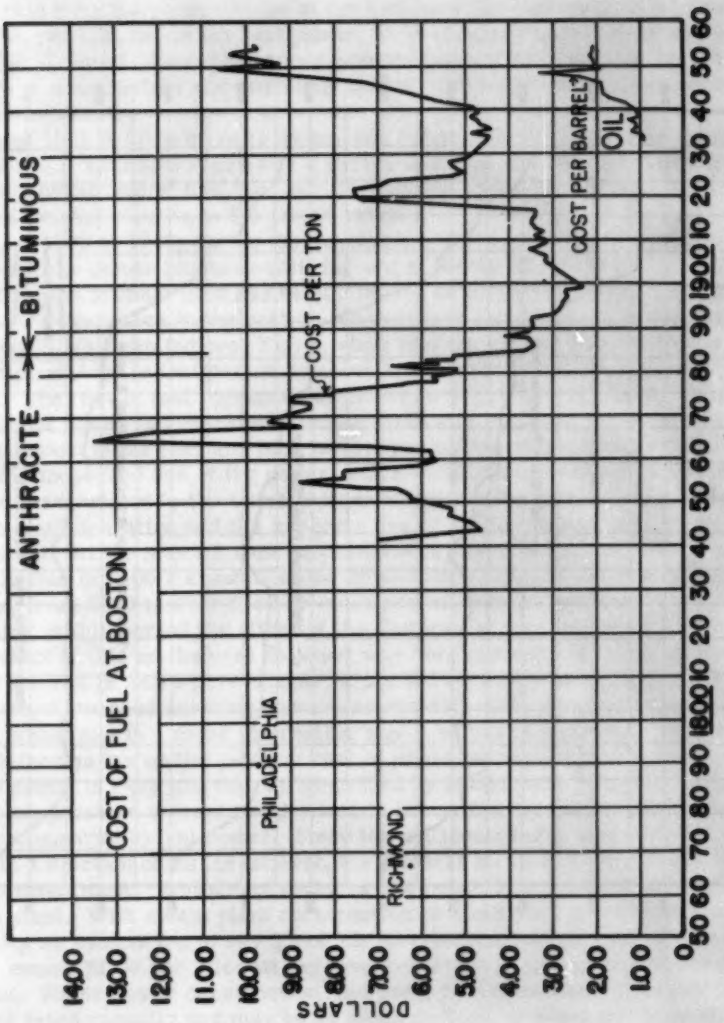

ANTHRACITE
BITUMINOUS
COST OF FUEL AT BOSTON
PHILADELPHIA
COST PER TON
RICHMOND
COST PER BARREL
OIL
DOLLARS
14.00
13.00
12.00
11.00
10.00
9.00
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0
50 60 70 80 90 1800 10 20 30 40 50 60 70 80 90 1900 10 20 30 40 50 60

Figure 6

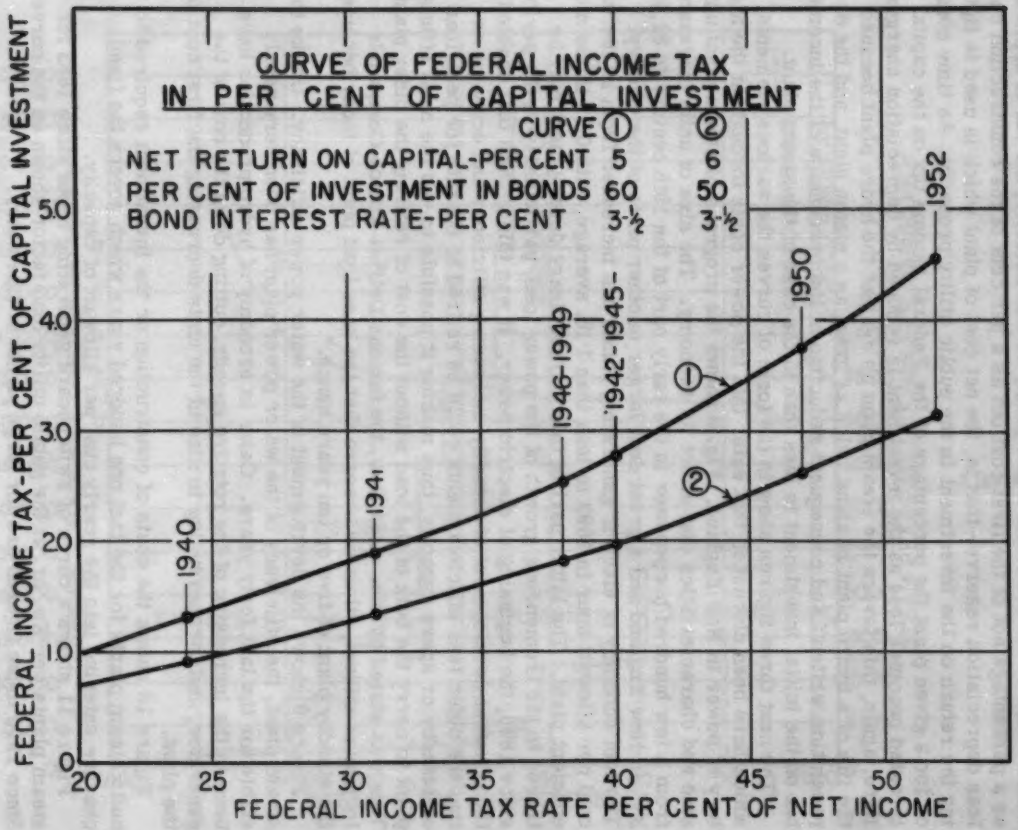

CURVE OF FEDERAL INCOME TAX
IN PER CENT OF CAPITAL INVESTMENT
CURVE ① ②
NET RETURN ON CAPITAL-PER CENT 5 6
PER CENT OF INVESTMENT IN BONDS 60 50
BOND INTEREST RATE-PER CENT 3-½ 3-½
FEDERAL INCOME TAX-PER CENT OF CAPITAL INVESTMENT
0
1.0
2.0
3.0
4.0
5.0
20
25
30
35
40
45
50
FEDERAL INCOME TAX RATE PER CENT OF NET INCOME
1940
1941
1946-1949
1942-1945-
1950
1952
①
②

Figure 7

tax adds for two cases of net return and the amount of bonded indebtedness (deductible before the income tax is figured). Actual income tax percentages paid by utility companies will not generally figure as high as these shown since the return on the investment is generally based on the net property values after depreciation, whereas these assume that the full cost is used, which would be the case with a new plant, so that actually, the tax is applied as a percentage not of the first cost but as a per cent of the construction cost less depreciation reserve—that is, the net cost of plant which is used in figuring the return on the investment in the public utility property. As time goes on for a given plant the percentage of the Federal income tax on the capital invested becomes less as the investment is reduced by depreciation charges. Here, again, this favors the steam plant as against the hydro plant because the life of a hydro plant is about twice as great as a steam plant, and the depreciation writeoff and consequent reduction of the percentage of the income tax on the initial investment is less than in the case of the steam plant.

The next three figures show in the form of curves the various elements which have been discussed that enter into the power costs throughout the history of power in New England. Fig. 8 shows the progress of steam plant unit size and characteristics that affect its economy. The size of units increases from a few hundred horsepower in the early part of the 19th century to 81,500 Kw. in New England and a great deal larger in other parts of the country. The fuel economy of electric generating plants has increased from 10 lbs. of coal per kilowatt hour in 1893 to less than 1 lb. average, with 0.82 in the most efficient plant. The latter part of this extraordinary development may be traced to the tremendous growth of the power load, particularly in the period since 1890, the beginning of electric power. It was this growth that enabled the constant development of steadily increasing efficiency and economy so that the older less efficient plants could be retired to operate for peak load or standby or spare capacity, thus making it possible to use the most efficient plant to carry the bulk of the load without the cost of retiring the older plants. This was stated by Mr. Alex Dow, the famous Detroit Edison Company's former president, "thus we observe that the base load plant of today becomes the standby plant of five or ten years hence."

Figure 9 shows the development of the water power efficiency. Unlike the steam plant, the efficiency of the water power plant has been very nearly stable for the last forty years. Gains in economy of hydro production have been in the increasing of the rotative speeds, cutting down the cost of the generator, and the increasing in size of the units decreasing the first cost of the plant.

Figure 10 shows the costs of construction or the investment required to build steam plants for the last one hundred years which govern the fixed charges entering into the yearly cost per kilowatt of capacity.

Figure 11 shows a curve of switchboard generating costs since 1883 for steam plants per Kw. hr. at the public utility load factor shown on the curve. Since 1910 these figures are for large plants at tide water.

Figure 12 is an attempt to show the comparative industrial power costs per kilowatt hour on the shaft or switchboard. In doing this the figures prior to the generation of electric power, which are figured in brake horsepower, are converted into kilowatts by straight proportion. The cost figures given are assumed to apply to industrial power in amounts of 500 and since 1910 also 2,000 Kw., the Kw. hr. figure being based on the normal operating hours prevailing in industry which are shown on the curve. During the period since

STEAM PLANT CHARACTERISTICS
1880
1900
1910
1920
1930
1940
1950
1960
0
90
50
30
20
10
40
TURBINES
ENGINES
STEAM PRESSURE
UNIT SIZE
EFFICIENCY
208 000 STATE LINE
260 000 DETROIT
300 000 AMERICAN
NEW ENGLAND
UNIT SIZE, KILOWATTS
EFFICIENCY, POUNDS OF COAL PER KILOWATT HOUR
STEAM PRESSURE, POUNDS PER SQUARE INCH

Figure 8

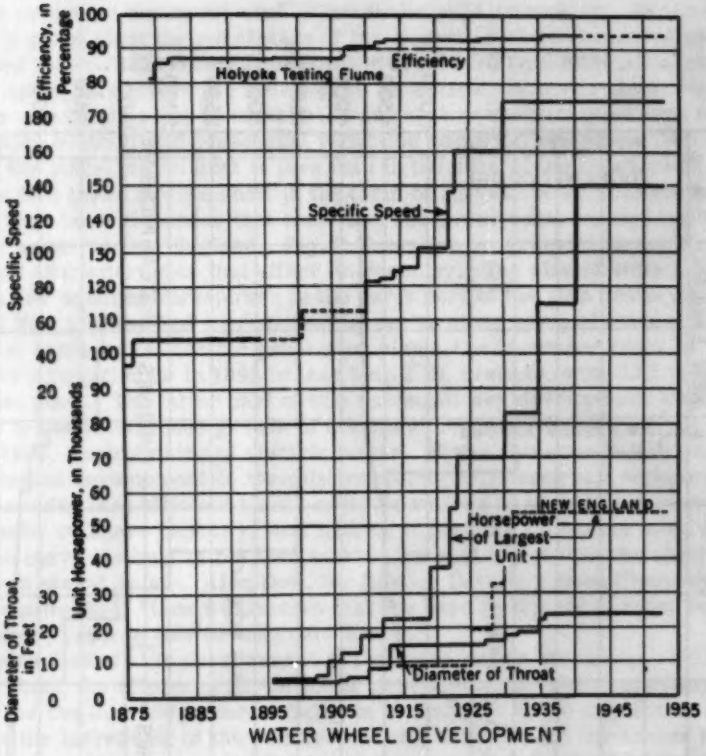

Efficiency, in Percentage
Specific Speed
Unit Horsepower, in Thousands
Diameter of Throat, in Feet
Holyoke Testing Flume
Efficiency
Specific Speed
NEW ENGLAND
Horsepower of Largest Unit
Diameter of Throat
1875
1885
1895
1905
1915
1925
1935
1945
1955
WATER WHEEL DEVELOPMENT

Figure 9

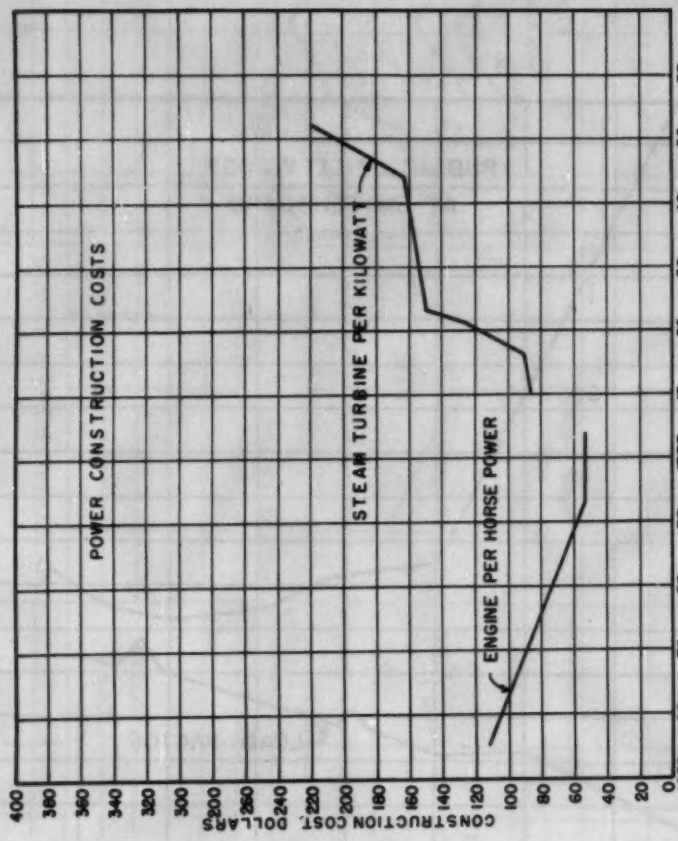
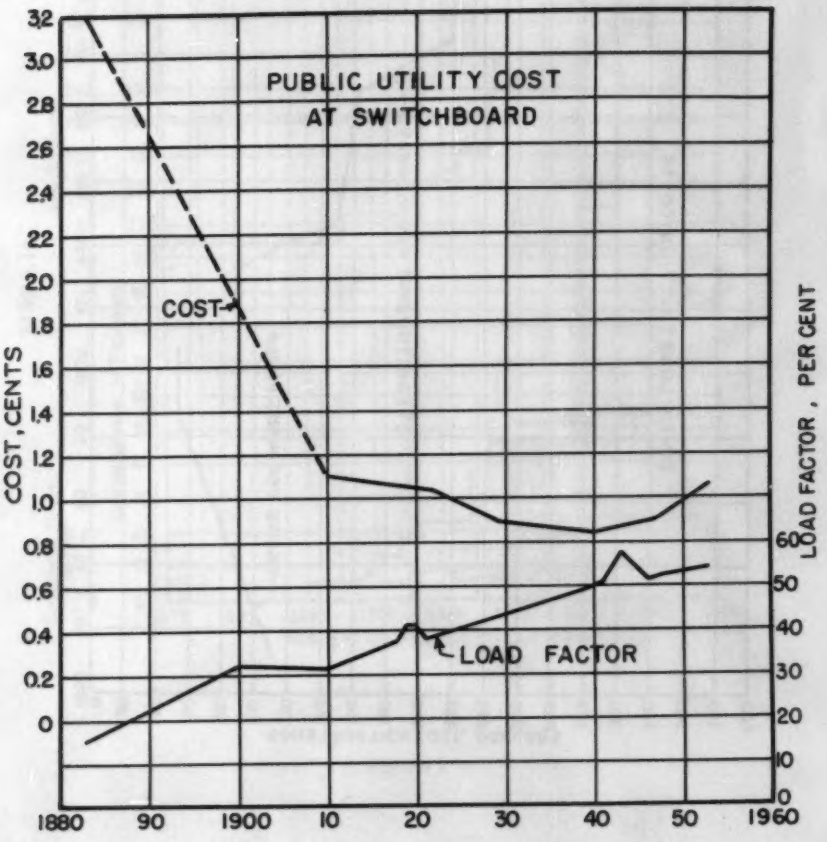

POWER CONSTRUCTION COSTS
STEAM TURBINE PER KILOWATT
ENGINE PER HORSE POWER
CONSTRUCTION COST, DOLLARS
400
380
360
340
320
300
280
260
240
220
200
180
160
140
120
100
80
60
40
20
0
1850
60
70
80
90
1900
10
20
30
40
50
60

Figure 10

Figure 11


PUBLIC UTILITY COST
AT SWITCHBOARD
COST
LOAD FACTOR
COST, CENTS
LOAD FACTOR, PER CENT
3.2
3.0
2.8
2.6
2.4
2.2
2.0
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0
60
50
40
30
20
10
0
1880
90
1900
10
20
30
40
50
1960

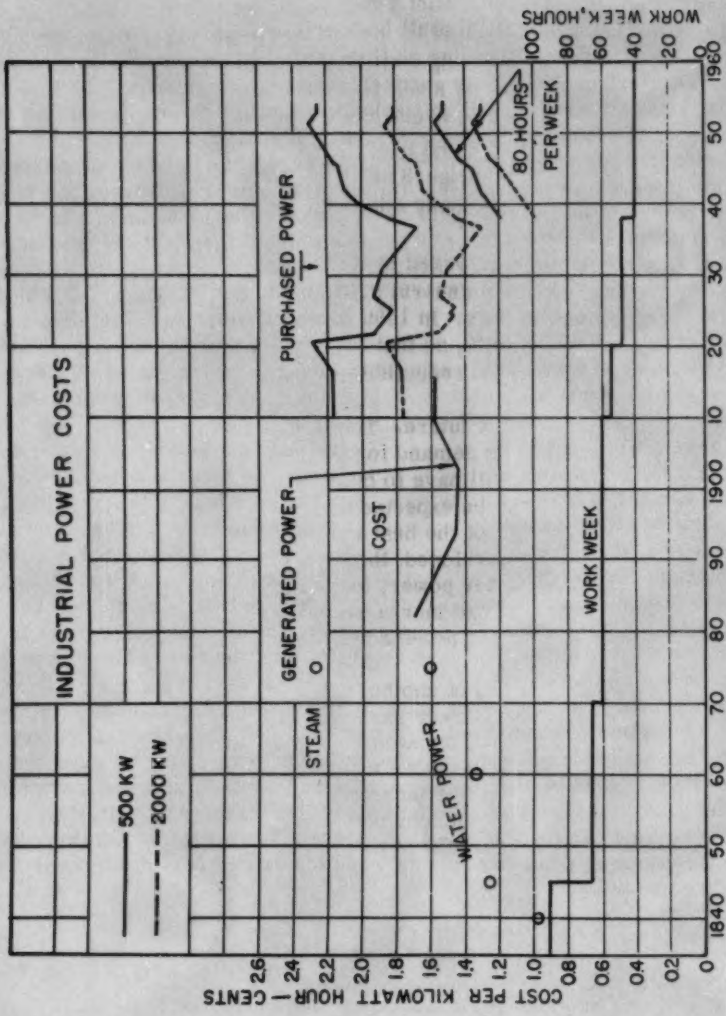

INDUSTRIAL POWER COSTS
500 KW
2000 KW
STEAM
GENERATED POWER
WATER POWER
COST
PURCHASED POWER
80 HOURS PER WEEK
WORK WEEK
COST PER KILOWATT HOUR — CENTS
WORK WEEK, HOURS
0
02
04
06
08
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
2.6
0
20
40
60
80
100
1840
50
60
70
80
90
1900
10
20
30
40
50
1960

Figure 12

1938 the 40-hr. week went into effect. The curve is given for these hours of operation to make a comparison on a cost basis with the past figures. However, general mill operation is on a two-shift or 80-hr. per week operating basis and the costs for this are also shown. Other hours of use such as a paper mill would show different results as would an industry using large amounts of process steam with power from exhaust steam.

It is interesting to note that in the period where accurate costs are available the power cost from the initial water power figures in 1840 which were about 0.98 to 1.17 cents per kilowatt hour had only risen from 1.55 to 2.25 cents per kilowatt hour depending on the hours of operation, despite an increase in construction costs by about five times. I have endeavored to describe in this paper how this remarkable result has been accomplished. The same thing, though, to a great deal greater extent of course, has taken place in the domestic lighting figure costs where there has been a large reduction in cost, but there are many other elements than just the cost of power that enter into these costs.

Figures such as these expressed in dollars do not, of course, tell us the full story. Fig. 13 shows the general price index which may be taken as the measure of the value of money. In 1850 this was about 66 and in 1953 about 320, an increase of about 500%, so that in effect, considering the true cost of power, there has been a great reduction in the cost of power in the past history.

History should point to the future. There does not seem to be any sign of a reduction in the increasing demand for power, so that the development of new plants, most of which will have to be other than hydro, will continue. Steam plant economy cannot be expected to increase much further along the present lines. Though most of the best water powers of New England, except some in Maine, have been developed, there will be room for the continual development of some more water power, including redevelopments, which may be found economical as the load increases to such an extent that there is room to use these undeveloped powers at low load factors on the extreme peak loads during low water.

We are probably at the end of another period—starting the use of atomic power. As the use of atomic power grows, and if it becomes more economical than fuel-generated power, the atomic power plants may be expected to fit into the electric load in just the same manner that each new and more efficient steam plant has fitted into the increased load in the past with the present plants becoming peak load or standby plants. The remaining water powers will be developed as peak-load plants when load conditions make them competitively economical.

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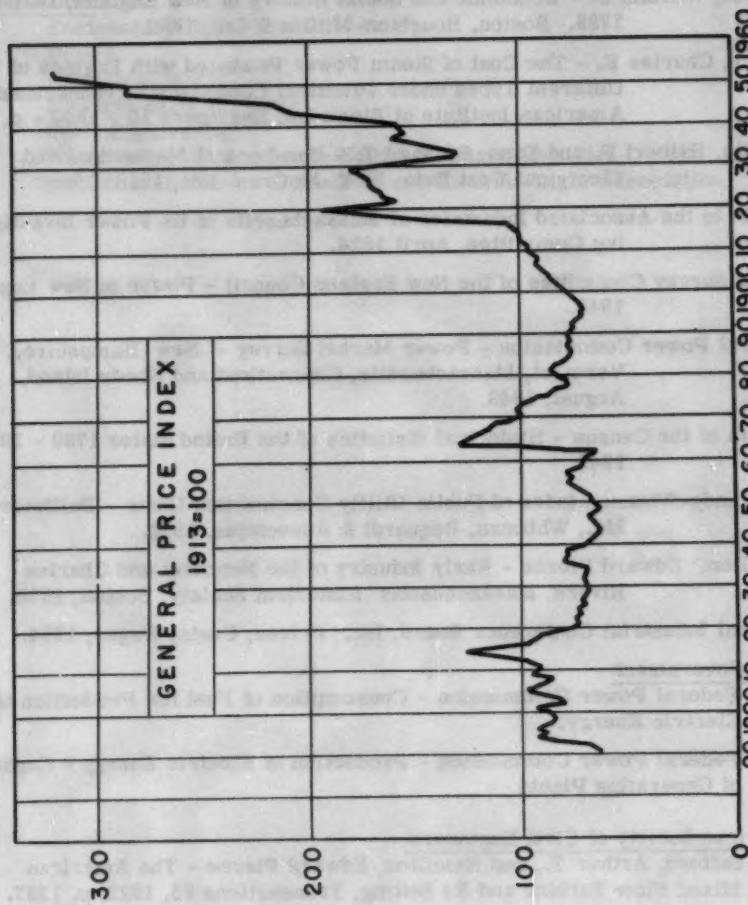
GENERAL PRICE INDEX
1913=100
300
200
100
0
90 1800 10 20 30 40 50 60 70 80 90 1900 10 20 30 40 50 1960

Figure 13

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Electric Light & Power.
Civil Engineering.
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Statistical Bulletins of the National Electric Light Association.

Paper 1044 PO 4

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

WATER SUPPLY FOR TEXAS STEAM ELECTRIC STATIONS

M. G. Salzman,* M. ASCE
(Proc. Paper 1044)

SYNOPSIS

Steadily increasing load demands and planning for new steam-electric generating capacity installations require a large amount of water for cooling purposes. The selection of suitable sites for new plants must be based on continuous and reliable water supply for present and future development. The available sources of surface and ground-water are used after extensive investigation of hydrological and geological aspects to determine safe yield and future decline of water levels. Where recirculation methods are employed, circulating water is reused with make-up required principally for evaporative losses. These methods include extended circulation in lakes and waterways, use of reservoirs on main streams as cooling ponds with storage capacity for conservation of flood waters, off-stream reservoirs with limited natural inflow and supplemental pumped make-up supply, cooling towers used for partial or complete recirculation of surface or ground-water supply. The requirements for municipal and industrial water supply have been combined with power plant needs by joint use of existing and new reservoirs with contractual agreements for coordinated operation and maximum overall benefit.

INTRODUCTION

Texas is richly endowed with natural resources, including diversified water supply. This water is essential for the agricultural, industrial development and the general welfare of the people and is largely responsible for the population growth and economic prosperity of this great State. The use of water as a cooling medium for steam-electric generation is a necessary function and contributes to the development or urban and rural communities, as well as industrial progress.

Water is indisputably the basic ingredient of our civilization as it exists today. It is not only required for human internal and external needs, but it also supplies moisture to the soil which yields material for food, clothing,

Note: Discussion open until January 1, 1957. Paper 1044 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 82, No. PO 4, August, 1956.

* Hydr. Engr., Ebasco Services, Inc., New York, N. Y.

and shelter, as well as the energy required to extract, process, fabricate and enjoy the products. It is apparent that there is a mutual dependency of water and energy production. The water requirements per unit of energy produced varies for different type plants with hydroelectric generating plants being the largest user. Steam and other thermal electric plants are next. Prospective nuclear fission plants are also potential users of this natural resource.

Industrial activity brings with it the need for electrical energy. Because of its large water requirements and the recognition of industrial and domestic needs the electric utility industry is "water conscious" and has endeavored to take an active part in the conservation methods to make more efficient use of the surface and ground-water required in the operation of its generating stations.

The author's experience in Texas has been largely with the use of water for cooling purposes in central steam electric generation. The client companies served include Dallas Power & Light Company, Texas Power & Light Company, Texas Electric Service Company and Houston Lighting and Power Company, which serve a large part of the State of Texas (approximately 78,000 sq. mi. of area).

The first three companies are components of the Texas Utilities Company and form the North Texas Interconnected System. The present net capability of the 15 steam-electric stations operated by the three companies is 1,828,000 kw and in order to keep pace with load demands 860,100 kw nameplate capacity has been installed in the last five years, more than doubling the capacity installed prior to January 1951. At present there are under construction extensions to three existing stations totalling 415,000 kw.

The Houston Lighting & Power Company serves the Houston Gulf Coast area, which is highly industrialized. The system load has almost trebled in the last ten years and at present the total capability in five steam-electric stations is 1,232,000 kw with two units at 165,000 kw each or 330,000 kw total capacity being installed in a new station scheduled for initial operation in 1956.

The problems involved in the use of water for these companies have varied with the geographic location of the generating stations, but in the main have been with respect to limited use because of remoteness of power load and fuel resources from large sources of water supply in rivers or coastal area. A previous paper presented at the Houston meeting of the Society in February 1951, described the influence of water supply in the location and layout of steam-electric plants in Texas and some of the problems involved.*

Sources of Water Supply

The source of water supply for steam-electric stations depends to a great extent upon the location of the plant. In many cases the availability of satisfactory quantity and quality of water is the motivating factor for the site selection, although fuel supply, electric transmission requirements and transportation facilities have a considerable influence. Economical comparisons must be made of alternative locations to determine the advisability of "bringing the plant to the water"or "bring-

* Type of Water Supply Influences Location and Layout of Texas Steam-electric Plants Published in "Civil Engineering" - May 1951 - M. G. Salzman and Louis Elliott - Members ASCE.

ing the water to the plant." All factors must be taken into account in this evaluation including tangible elements upon which investment dollars can be placed and intangible items, such as area development, highway and bridge restrictions, encumbrances of pipe line crossings and problems of land acquisition. The ultimate planned capacity requires foresight as to the future dependability of fuel and water resources and a long range development program must be made. For this reason sites are selected which will permit periodic installation of additional units of increasing size, perhaps to a presently foreseeable maximum plant capability of 1,000,000 kw.

The continued availability of water for successive and final installation is of utmost importance. The ultimate requirements of water for condensing and other plant uses for a direct condensing plant may total more than 1,000 cfs. Obviously, there are not many rivers or locations except along tide water where this quantity of water is available on a continuous basis. For this reason, recirculation or reuse of water is necessary so that maximum capacity can be installed with minimum water use.

An analysis of the annual rainfall of Texas indicates that sufficient water falls on the area to meet not only all present needs but more than three times as much. Despite this fact, there have been serious shortages of municipal supply in certain areas which have necessitated rationing of water during the past drought periods. This condition has resulted primarily from seasonal variation and unequal distribution of the rainfall and from the lack of sufficient storage facilities and conservation methods to provide effective utilization of the available water.

The interception of surface and ground-water concerns the engineer and it is the application of his knowledge of the factors involved which determines the efficiency of utilization. Much useful work has been stimulated by Federal and State agencies in cooperation with the agricultural and industrial interests to conserve water through reservoir development and soil erosion control in the drainage areas. The utility companies mentioned have shown increasing interest in reservoir development as indicated by the fact that eight reservoirs have been constructed for cooling ponds in their service areas where only two existed before World War II. In addition at least four more are presently contemplated.

Ground-water in many parts of Texas represents the major source of water supply for both domestic and industrial consumption. This type of water is particularly attractive to utilities since it is relatively constant in chemical character and temperature and contains less suspended matter and is more apt to be free from contamination by sanitary and industrial waste. However, ground-water often contains higher concentrations of soluble salts than surface water.

Water Requirements of Steam-Electric Stations

Steam-electric power plants, whether used to supply energy to an industrial establishment or as a central station to feed into a transmission network, use water for various purposes. The largest portion of the water is required for condensing steam. Cool water is circulated through the condenser removing heat from the steam to produce a vacuum and increase turbine efficiency. To most effectively accomplish this task it must enter in adequate quantity and at low temperature. After passing through the condenser the heat laden

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water may be cooled and reused again or, where adequate incoming cool water is available, it is discharged to the stream or large body of water.

The water used for cooling represents approximately 95 per cent of the total plant needs. For a modern high efficiency installation, the amount of condenser circulating water varies between 120 and 200 cfs per 100,000 kw of generating capability, depending upon the turbine throttle steam conditions, the thermal cycle, the number of feedwater heaters and the condenser design. The remaining five per cent of the plant water requirements is for cooling of a variety of equipment, for boiler feedwater make-up and for general service and sanitary use.

It should be pointed out that although the plant circulating water requirement may be a large volume of water, there is usually only a relatively small portion of this water no longer available for further use by either the power plant or other users. This is the amount which is dissipated by increase in evaporation losses after the water is discharged from the condenser. The increase in evaporation losses results from the greater temperature differential existing between the water and the air than would naturally occur. The amount of water evaporated is not a fixed quantity per kilowatt or a fixed per cent of the circulating water requirement since it is dependent upon the temperature rise of the body of water into which it is returned. For various atmospheric conditions and temperature rise above the natural water temperature, the increase in rate of evaporation loss can be estimated. This is sometimes called "forced evaporation" to differentiate from natural evaporation which would take place without heat entry; however, the same climatological data and methods of heat transfer from water surface to air are used to determine these evaporative losses.

Development and Methods of Use

A power plant located on a large lake, river or bay may be considered to have a "once-through" or direct-condensing system. This means that for all practical purposes, the water which passes through the condenser will not return again. Recirculation of the discharged water may occur in the case of a plant having an adequate water supply because of dams constructed downstream below a river plant or the effect of tides or insufficient separation of the intake and discharge structures. These cases of recirculation usually occur only for a short period and they may still be classified as direct-condensing systems.

Recirculation systems generally apply to those types of circulating systems which reuse all or a portion of the water passed through the condensers and are in three groups, namely;

1) Cooling ponds
2) Cooling towers
3) Extended circulation

These groups may be further subdivided depending upon source of make-up water supply and whether or not they are connected directly or indirectly with another type of circulating system.

The variety of circulating water systems available for power plant use lends some degree of flexibility to plant site selection. However, for the most part the peculiarities of each system dictates which one can be adapted

to a particular site. This becomes evident if it is borne in mind that cooling ponds need considerable land area, cooling towers are usually justified only when the site results in substantial savings in either fuel cost or transmission, and extended recirculation requires minimum stream flows equal to more than fifty per cent of the plant needs. More often the problem is not one of circulating water system selection but one of developing the plant in a manner which will result in the maximum utilization of the site.

Cooling Ponds

Cooling ponds are a means of providing water which can be recirculated and used on a continuous basis with depletion of storage capacity only as a result of evaporation due to atmospheric conditions, temperature rise resulting from the heat rejected in the condensing process and possible seepage or gate leakage losses. The amount of heat to be dissipated is variable and proportional to the steam conditions of the units and amount of energy generated. The natural water temperature, surface area and depth of the pond require careful analysis and design of structures should provide for adequate separation of intake and discharge for efficient utilization of the water volume.

Where a large lake or reservoir is already available, the existing water supply can be made to accomplish double duty and provide a cooling water source in addition to its original purpose. In most cases, however, it is necessary to construct an artificial pond either on the main stream or a tributary creek for storage of water during high flow periods and drawdown of the pond during dry periods. A cooling pond supplied by run-off obtained from a considerable drainage area and stored seasonally for a period of years is called an "on-stream reservoir." Drawdown may be to the extent necessary to retain a residual pond of adequate depth and surface area. The configuration of the lake and the availability of sufficient area and depth under the extreme conditions of maximum drawdown during a prolonged dry period, determines the capability of the pond and the amount of generating capacity which can be safely installed.

The following are some of the major problems which must be considered in the use of "on-stream reservoirs."

1) Sufficient inflow during drought periods to replenish depleted storage capacity resulting from natural evaporation, evaporation caused by heat dissipation, seepage and water use for other than cooling.
2) An adequate spillway designed to pass the maximum probable flood and control the lake level so that excessive flood rise will not inundate the plant area or cause damage to upstream or downstream property.
3) Allowance in the residual pond for loss of capacity due to siltation from erosion of the watershed.
4) The necessity for the purchase or easement of a large land area to provide for high water level.
5) The relatively high initial investment required for the land and reservoir structures necessary for the ultimate plant capacity to be installed.

Because much of the effectiveness of cooling ponds results from the availability of exposed surface area for heat dissipation with only small depletion of storage resulting from the heat input, it is possible to employ some of the larger storage reservoirs for multi-purpose use as a cooling pond and to provide for domestic and industrial water supply. This combined use

results in lower initial investment costs for the utility and water district or municipality with maximum overall benefit.

While on-stream reservoirs with ample storage capacity for prolonged dry periods are effective for large capacity installations there is also the opportunity to develop smaller off-stream cooling ponds on tributaries or in low areas adjacent to the large streams. In this case, the pond will have reduced surface area and volume with natural inflow from direct rainfall and small area drainage to keep it full during normal or wet years, but during dry years replenishment of storage depletion most come from pumped make-up. Pumping stations are built with ample capacity to take advantage of high flows and use of purchased water from other storage reservoirs or river flow not required by prior downstream appropriators. Pumping must usually be done in the non-irrigation season or at such time when the river is in flood stage.

Off-stream ponds may have the following advantages over "on-stream" reservoir developments:

1) The effective surface area and consequent land acquisition may be substantially reduced.
2) Siltation is minimized since there is no large drainage area involved.
3) The spillway will have greatly reduced capacity because of the smaller watershed drainage.
4) Unless extensive diking is required or pump make-up lines are long, the investment cost for circulating water structures will be lower than for on-stream reservoirs; however, if supplementary make-up water must be purchased annual costs may be higher.

The development of a cooling pond, either on-stream or off-stream requires considerable analysis of the hydrological factors to determine the yield of the watershed and capability of the reservoir. As most of the cooling ponds are located on relatively small drainage areas on tributary streams, there is usually a lack of gaging stations for definite stream flow data and estimates of the run-off must be made using records from other nearby streams or rainfall data. This should be compiled for as long a period as possible to determine the historical record of droughts and safe yield during the most critical period. Expectable minimum and normal year inflow can thus be established to determine pond replenishment.

Reservoir losses, including natural and forced evaporation, seepage and withdrawals for other than cooling must be estimated. Where records of evaporation pans located on lakes subjected to similar atmospheric conditions are available, these may be used with suitable correction factors to apply to the pond under consideration. Considerable work has been done by the Texas Agricultural Experiment Station in cooperation with the Texas Board of Water Engineers and the U. S. Department of Agriculture, in correlating this information in published form and the records of the U. S. Weather Bureau are valuable data for these estimates.

Cooling Towers

Cooling towers play an important part in the development of many sites where cooling ponds of adequate size are not feasible for recirculation. The use of cooling towers avoids the necessity of obtaining extensive land area for reservoirs and appropriation of the water necessary for filling and replenishing cooling ponds. A cooling tower site may be built for a lower initial cost than a cooling pond development as the circulating water and make-up

facilities need to be provided only for the capacity initially installed. The investment in water facilities for future units is, consequently, deferred until needed. After the second or third extension is made the cost of towers may offset the cost of the cooling pond and for the ultimate plant the economies usually favor the pond development. Each situation, however, must be considered on its own merits.

Some of the problems to be considered in the use of cooling towers are:

1) A generating capability loss with reduced net output occurs because of the increased auxiliary power requirements necessary to operate fans for induced or forced draft towers.
2) The service life of cooling towers is relatively short because of deterioration from alternate wetting and drying of wood filler and concentration of chemicals.
3) Additional make-up water is required for blowdown and drift losses caused by water mist entrained in the circulating air.
4) The possibility of objectionable fog conditions to surrounding area.
5) The increased cost of water treatment necessary to provide satisfactory water for recirculation in the tower.

Extended Circulation Systems

Situations arise where water is available from small rivers, bayous and/or tidal basins in which the quantity of water is inadequate for the ultimate needs of the plant and water surface area in the immediate plant vicinity is insufficient for surface cooling. In these cases, it has been proved advantageous at times to construct either long intake or discharge canals or both in an effort to provide some mixing of the warm discharge water with the cooler natural flow and to obtain some degree of surface cooling.

Extended circulation on a river requires the benefit of a downstream dam to cause recirculation of the discharge water upstream again. In this case, the major benefit is derived from the mixing of the warm discharge and the natural river flow. Very little benefit results from surface cooling owing to the small area generally associated with the discharge canal and river.

As a result of the meandering path of many bayous and their interconnection with other water bodies such as rivers and canals, it is possible by careful selection of discharge and intake canal layout, to obtain an exceedingly long recirculation path. The surface area obtained from this long path in conjunction with many small inflows of cool water, will in some cases materially reduce the temperature of the discharge water before returning to the plant. If the bayou and/or its connecting water bodies are effected by tidal variations, the benefits derived from extended circulation are materially increased.

In almost all cases of extended circulation, considerable lengths of canal are necessary sometimes ranging up to four or five miles. Unfortunately due to the small surface area in these canals, there is a very small amount of cooling but considerable expense for land and excavation. Therefore, careful study must be made of the cooling effect of the bodies of water into which the circulating water is eventually discharged.

Application of Circulating Water Systems at Typical Stations

In Texas, the circulating water systems are principally cooling ponds and cooling towers except along the coastal area where the use of tide water from

either the Gulf of Mexico, ship channels or estuaries permits direct condensing with extended circulation. To consider a direct condensing system inland is not practical since there are no rivers in Texas having sufficient year round flow to meet the needs of the size plant contemplated in the modern electric system.

In many areas cooling tower installations using ground water exclusively, from either shallow or deep wells, are reaching a limit to their economical expansion because of lowering water levels. Cooling towers obtaining their make-up from both ground-water and storage ponds have been used to provide some relief to this situation.

It is apparent that recirculating water systems utilizing surface water should be employed wherever possible. The development of reservoirs for storage of flood water aids the cause of water conservation and augments the water supply for the adjoining area by providing additional storage for the flood waters released from upstream reservoirs. In most cases the water required to fill cooling ponds or cooling tower make-up storage ponds and to replenish losses due to natural and forced evaporation and seepage can be obtained from local drainage and flood waters. However, there are some cooling ponds which require "firming-up" by supplementary make-up from either wells, purchased water or releases from upstream storage reservoirs during droughts.

One of the earliest examples of a reservoir developed by a utility to provide an adequate supply of condenser cooling water is the Mountain Creek Reservoir serving the Mountain Creek Station of Dallas Power & Light Company. This reservoir is dependent entirely upon the 300 square mile drainage area above the reservoir. With the completion of the present extension, approximately 354,000 kw will be installed on the 2,800 acre lake. Over the course of 20 years that this reservoir has been in operation there has been considerable silt deposited in the lake. This has occurred principally at the upper end where surface cooling is not affected. Storage volume has been reduced, however, and to retard further siltation resulting from soil erosion in the drainage area, Dallas Power & Light Company has energetically cooperated in a soil conservation program with the upstream land owners and the results have been beneficial to all parties. Figure 1 shows construction of new 100,000 kw installation with discharge canal extended into the lake for effective use of lake surface area.

In some instances, lakes have been or are being developed for the joint use by the utility as a cooling pond and by nearby cities or water conservation districts as a domestic water supply with provision for sale of excess water to local industries. Contractual arrangements have been made to safeguard the interests of all parties concerned and to aid in financing the project. These contracts usually prescribe a definite portion of the safe yield of the reservoir for power plant use and a minimum and maximum pond level to maintain adequate cooling surface area.

Figure 2 is an aerial view of the lake constructed at Colorado City in West Texas, by Texas Electric Service Company for combined use as a cooling pond for the Morgan Creek Steam-Electric Station and municipal water supply for Colorado City. At present 150,000 kw generating capacity is installed at this location. The drainage area is 274 square miles and the storage capacity of the lake is 31,000 acre-ft. with a surface area at full pond about 1,600 acres. A thermal study has been undertaken on this lake in cooperation with the U. S. Geological Survey to determine the effect of heat entry

on the total evaporation rate. An evaporation station was established at the plant with measurements made by plant operators of evaporation, wind velocity and air temperatures. Lake level variations are observed and temperature effects determined at various locations on the lake by floating thermometers to obtain daily maximum and minimum values. Stream gages were also established to measure incoming water supply. The method used is a modification of the so-called "energy budget" method developed by the USGS and other Federal agencies at Lake Hefner in Oklahoma to measure the energy conducted to and from the water as sensible heat including solar radiation and reflected radiation.

Figure 3 illustrates an off-stream cooling pond is the Lake Creek station of Texas Power & Light Company which is located approximately 12 miles south of Waco, Texas. This lake was formed by a dam near the outlet of Manos Creek, a small tributary of the Brazos River, which drains about 15-1/2 square miles. During cloudbursts and freshets there is considerable inflow to the pond but most of the storage replenishment is obtained by pumping from the Brazos River under appropriation rights granted by the Texas Board of Water Engineers. The lake has a surface area at full pond of 550 acres and the total storage capacity of the lake is 8,500 acre-ft. A single 65,600 kw generating unit is presently installed and there is provision for future installation of additional units.

Figure 4 shows the cooling pond for the River Crest plant of Texas Power & Light Company is impounded in a diked area adjacent to Sulphur River in north central Texas. River supply may be pumped to replenish the 560 acre pond surface area during dry periods since there is practically no local drainage. Because this pond is located in a region where average annual rainfall exceeds evaporation, there is normally sufficient water available in the pond without pumping. At present an 80,000 kw unit is installed with provision for at least three extensions.

The water supply for most of the cooling tower plants in Texas is obtained from ground water sources, although river water make-up may be used. These plants are generally located near metropolitan areas and the use of the same source of water by industries and minicipalities requires careful analysis of prospective pumping levels. Preliminary investigation by drilling test holes, and pumping tests on developed wells are necessary to determine transmissibility, and storage characteristics to determine the safe yield of the aquifer over a prolonged period. A study of the geological features of the area, including recharge, depth and extent of water bearing formations and present and potential use of the underground supply enables establishment of optimum pumping rates and well spacings. The use of electrical prospecting methods and logs of existing wells provide valuable assistance in these investigations.

In the Dallas-Fort Worth area the several formations of the Trinity Sands supply ground-water which is used as cooling tower make-up at several steam-electric stations in this area, including the Handley Plant of Texas Electric Service Company in Fort Worth, Dallas and Parkdale Stations of Dallas Power & Light Company in Dallas and Collin Station of Texas Power & Light Company, about 30 miles north of Dallas. The dip of these strata is southeasterly with thickening of the water bearing strata as they progress. As a result the Fort Worth wells can obtain satisfactory ground-water from the 1,000 ft. depth but in the Dallas area the wells are drilled 2,800 to 3,200 ft. deep for quantities in excess of 1,000 gpm.

Figure 5 is a map and geological section showing recharge area, incline and variable thickness of the principal strata of the Cretaceous series of Trinity sands. The drought conditions which prevailed during the past five years, along with the increased use for domestic and industrial purposes, have imposed a severe drain upon the ground-water supply and the general artesian water table has declined considerably. For this reason several of the plants have located well fields some distance away from the area of heavy pumping and are seeking sources of surface supply which can augment the ground-water supply. The Parkdale Plant of Dallas Power & Light Company has recently put into service a small 80 acre off-channel storage pond which can be used to store surplus water pumped from White Rock Creek. Ground-water drawdown is thereby relieved and the storage pond serves as an emergency supply when a well is out of service. Figure 6 is an aerial view of the pond taken during initial filling operations.

The Webster Steam-Electric Station of Houston Lighting & Power Company is an example of a plant having an extended circulation system. The primary source of water is Clear Lake which is a tidal water body connected with Galveston Bay. The circulating water system consists of a 20,738 ft. discharge canal terminating at Clear Lake. Clear Creek, which naturally flows into the western end of Clear Lake about 3,500 ft. from the point of discharge canal outflow, provides a means of recirculation back to the plant intake located about 3.5 miles upstream. Total path of circulation, including the 3,750 ft. intake canal, is 8.74 miles, neglecting any dispersion of the warm discharge water over the lake surface. Figure 7 is a plan of the circulating water system indicating tidal effects and use of existing waterways and Figure 8 is an aerial view showing the intake and discharge canals.

Houston Lighting & Power Company in an effort to ascertain how much of the lake is used for surface heat dissipation, has installed six temperature recorders at strategic locations. The data obtained to date cover a two year period before the plant went into service and approximately a year and a half with the plant in operation. These data are of particular interest because of the shallowness of the lake (between 2 to 10 ft. deep) and the effect of the tides on lake cooling performance. Over an extended period of time these data will prove extremely helpful in determining the possible ultimate development of the plant.

CONCLUSION

The utility companies in Texas have made diligent efforts to cooperate in all water projects which aim towards more prudent use of presently available surface and ground-water supplies. They have also given much thought and study to the use of other possible water sources as conservation measures. The use of treated sewage effluent is one method whereby water which has served one purpose can be effectively reused. Demineralization of sea water has received considerable attention and although present production costs are quite high, it is anticipated that some day this may provide a satisfactory means of conserving fresh water supply.

The industry has followed the trend to more efficient use of existing water resources above ground and underground by adopting measures wherever possible for reuse and conservation of waste water. To meet the future challenge of increased demand by expanding cities and industries, it is essential to

obtain the cooperative effort of all interested parties to develop new sources and provide joint use of facilities for maximum overall benefit.

ACKNOWLEDGMENT

The author expresses appreciation to the operating companies mentioned in this paper for their cooperation in making available the illustrations of the typical circulating water systems at their plants. These represent only a few of the many other plants on their systems which employ novel features for effective utilization of available water supply. Acknowledgment is also made for the advice and assistance rendered by associates in Ebasco Services in the preparation of this paper.

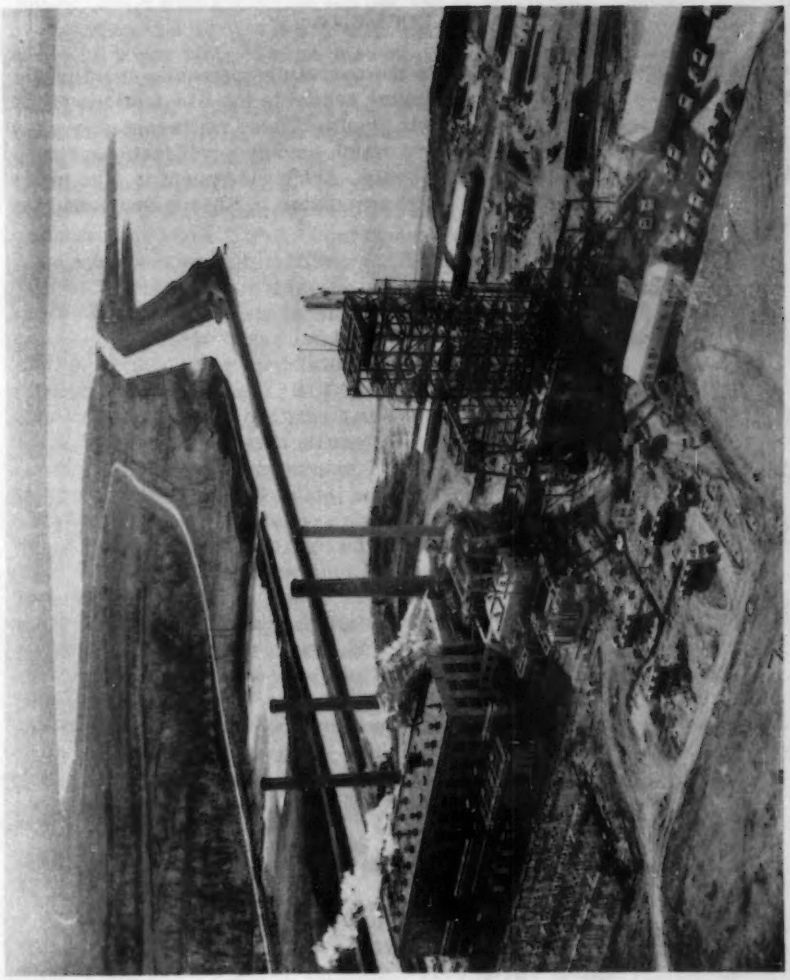


Figure 1 - Mountain Creek Lake - Dallas Power & Light Company
View of plant extension and lengthened discharge canal

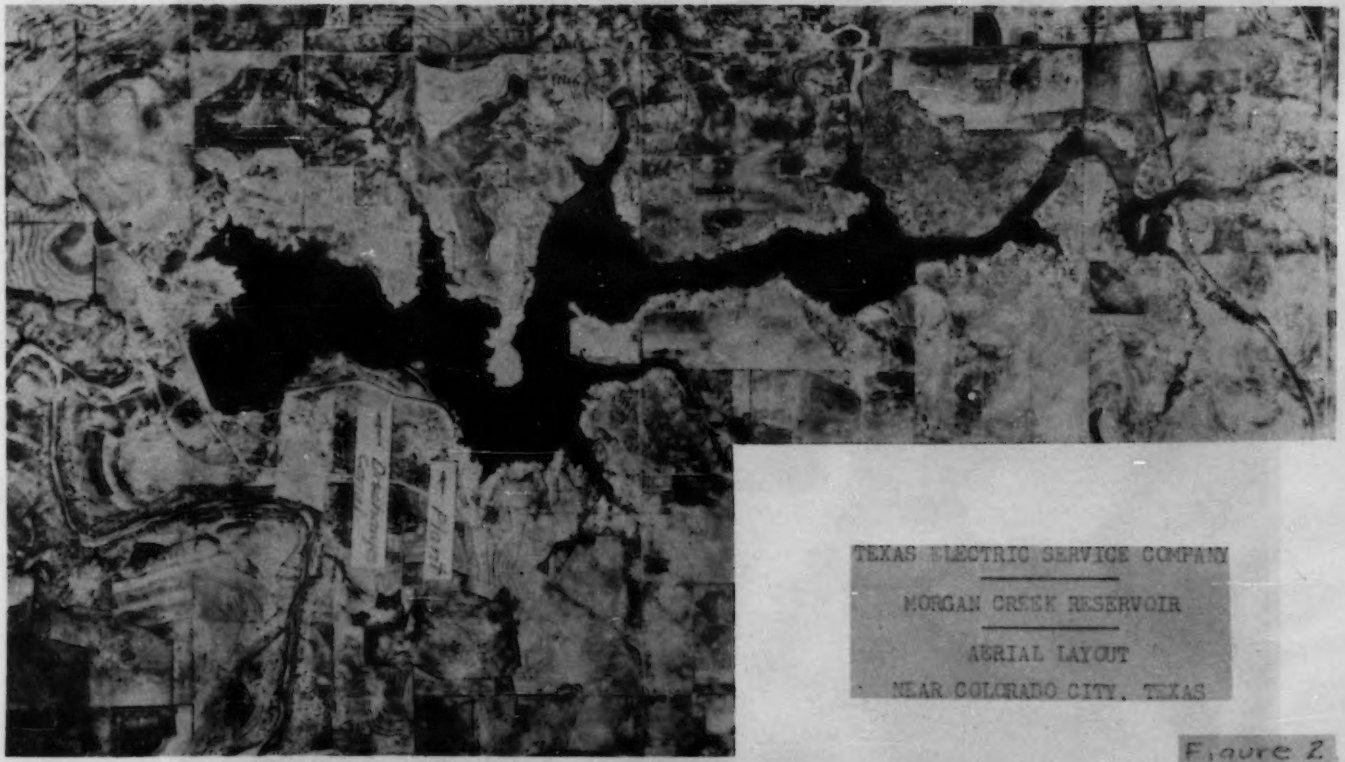


Figure 2 - Lake Colorado City - Texas Electric Service Company
Aerial layout of Morgan Creek Plant and Reservoir

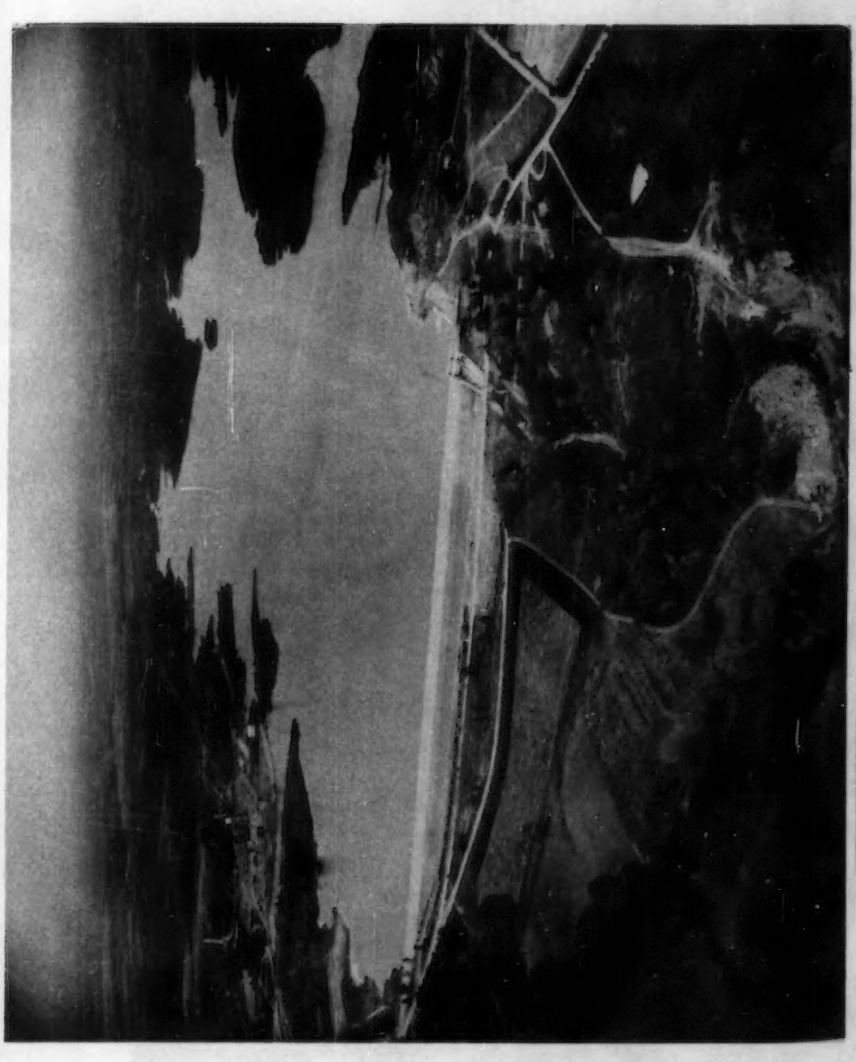


Figure 3 - Lake Creek - Texas Power & Light Company
View of lake, station and make-up pumping plant

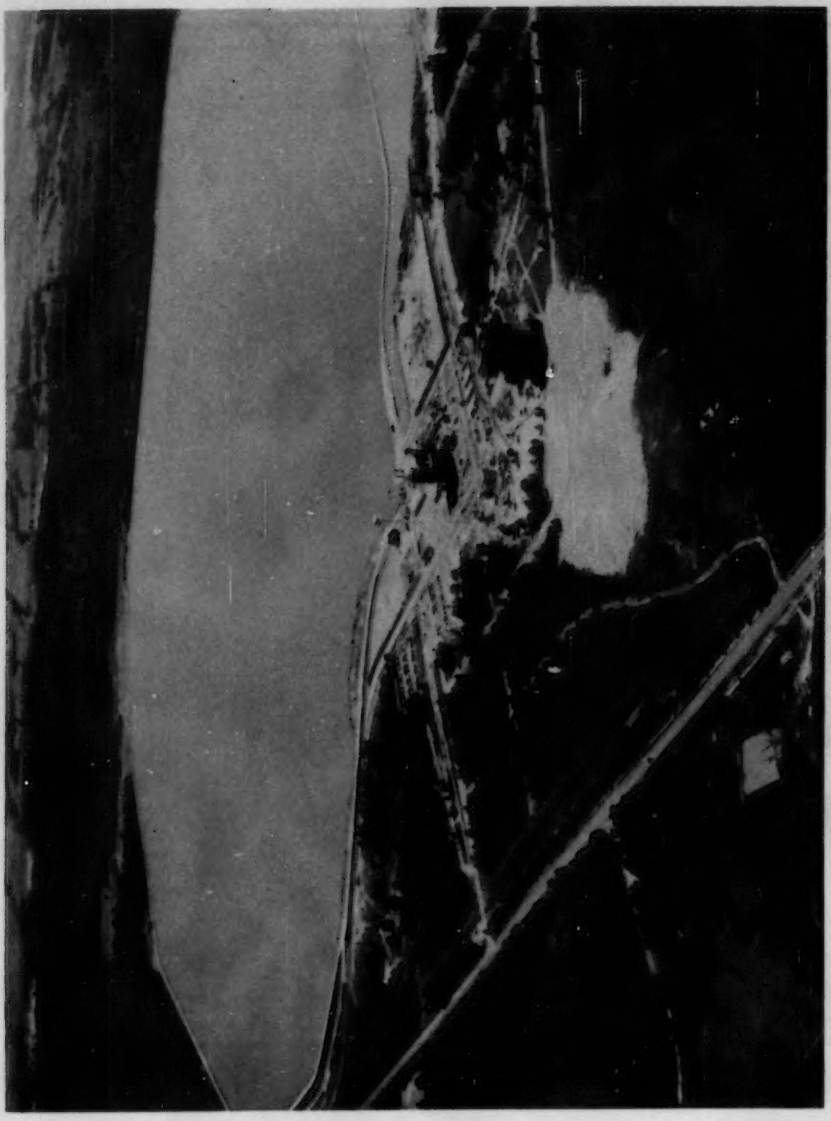


Figure 4 - River Crest - Texas Power & Light Company
View of cooling pond and generating station

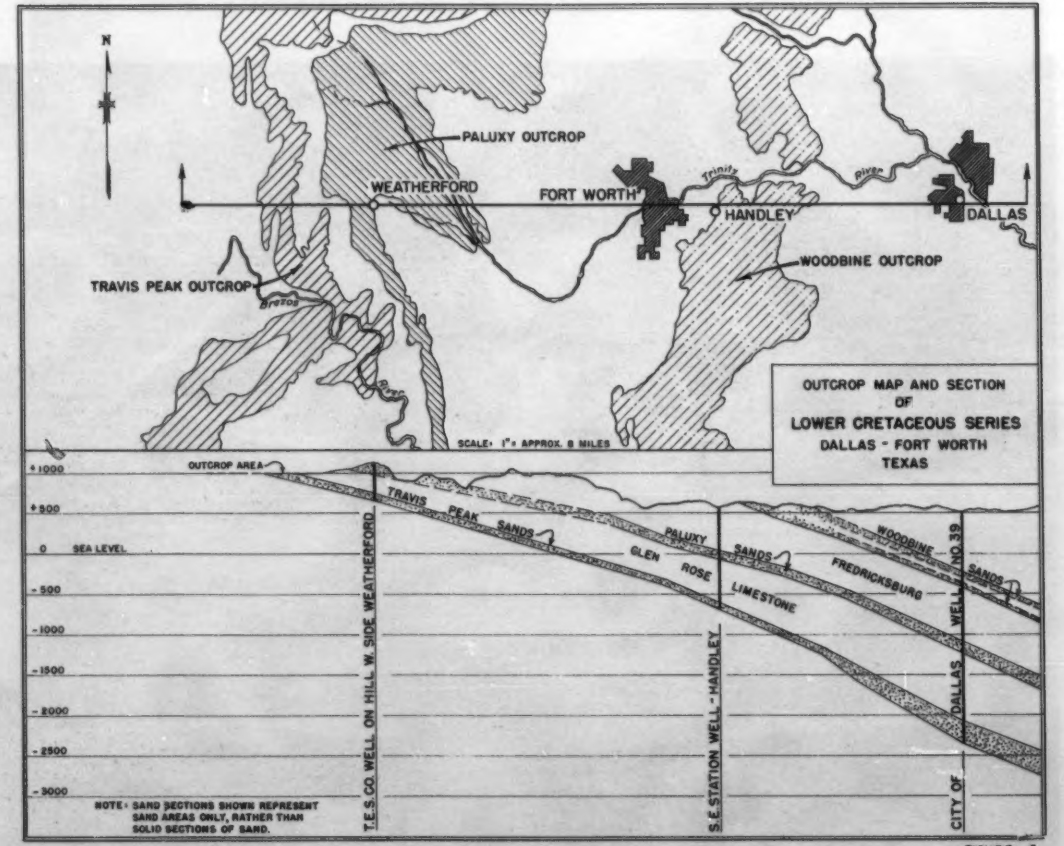

N
PALUXY OUTCROP
WEATHERFORD
FORT WORTH
Trinity River
HANDLEY
DALLAS
WOODBINE OUTCROP
TRAVIS PEAK OUTCROP
Brazos River
SCALE: 1" = APPROX. 8 MILES
OUTCROP MAP AND SECTION OF
LOWER CRETACEOUS SERIES
DALLAS - FORT WORTH
TEXAS
+1000
+500
0 SEA LEVEL
-500
-1000
-1500
-2000
-2500
-3000
OUTCROP AREA
TRAVIS PEAK SANDS
PALUXY SANDS
GLEN ROSE LIMESTONE
FREDRICKSBURG
WOODBINE SANDS
T.E.S.CO. WELL ON HILL W. SIDE WEATHERFORD
S.E. STATION WELL - HANDLEY
CITY OF DALLAS WELL NO. 39
NOTE: SAND SECTIONS SHOWN REPRESENT SAND AREAS ONLY, RATHER THAN SOLID SECTIONS OF SAND.

FIGURE 5

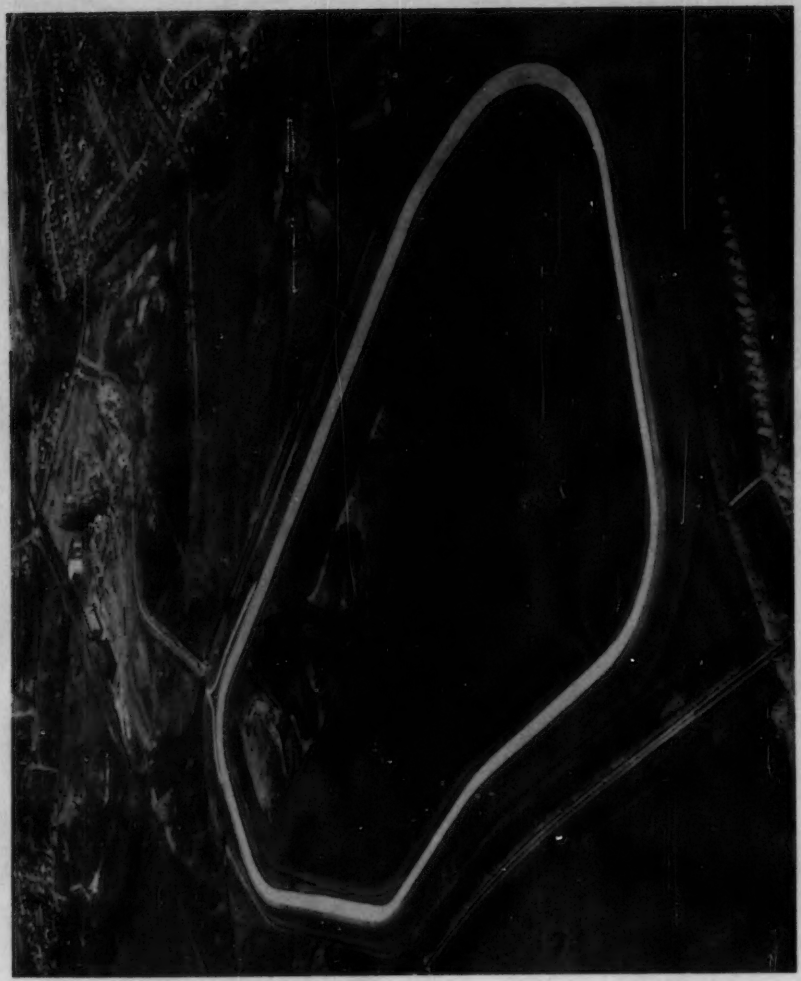


Figure 6 - Parkdale Steam Electric Station - Dallas Power & Light Company
Aerial View of make-up storage pond for cooling-towers

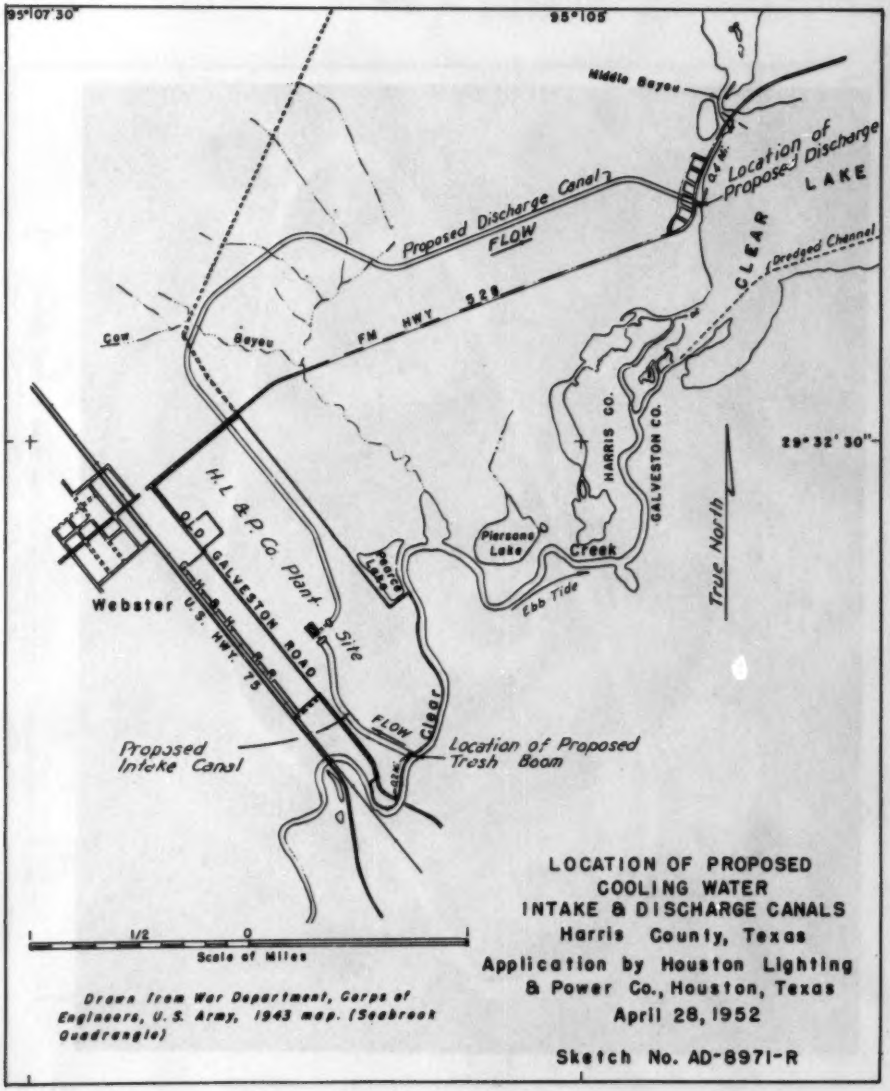


Figure 7 - Webster Steam Electric Station - Houston Lighting & Power Company
Plan of circulating water system

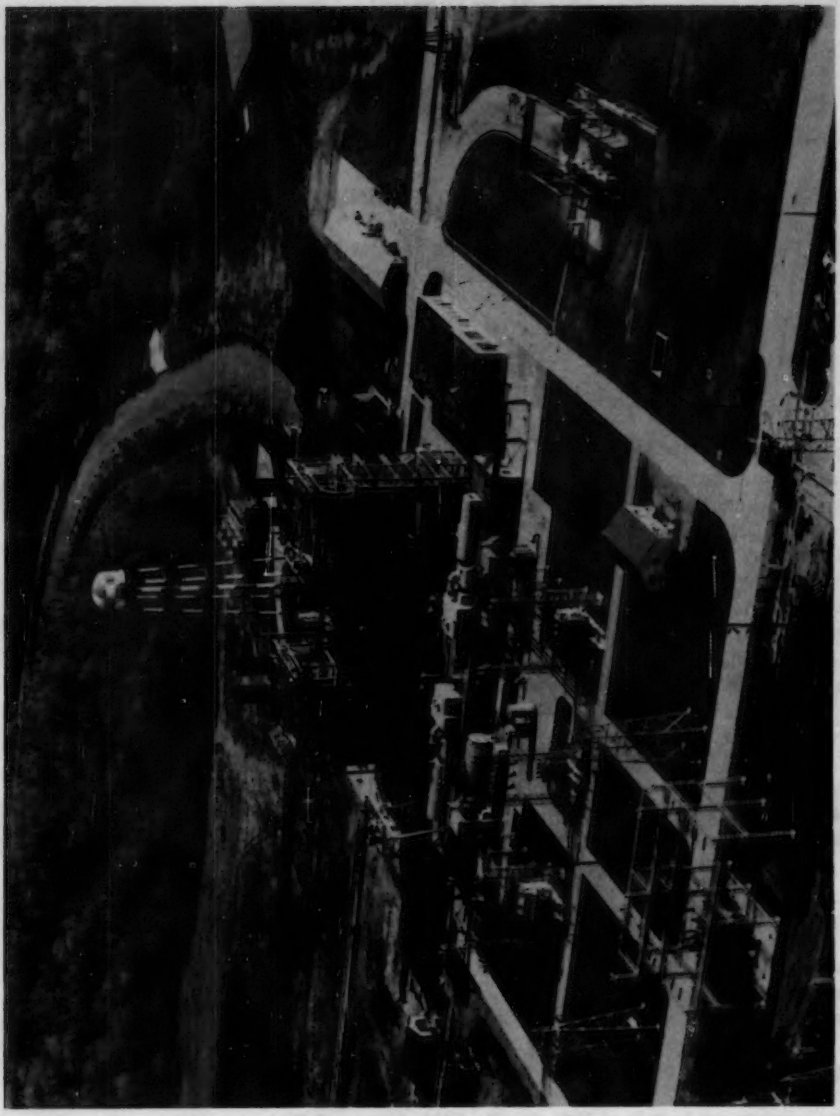
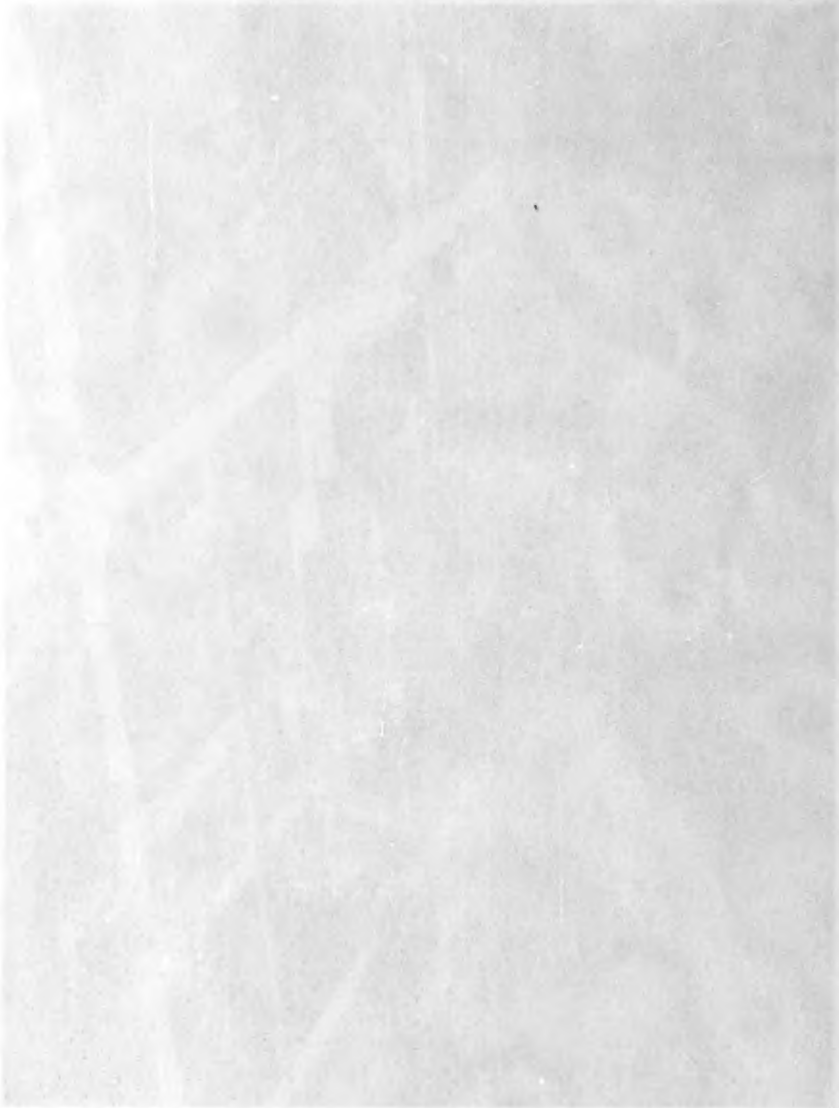


Figure 8 - Webster Steam Electric Station - Houston Lighting & Power Company
View of plant showing portions of intake and discharge canals



Paper 1045 PO 4

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

ARCH DAMS: DESIGN AND CONSTRUCTION OF ROSS DAM

C. E. Shevling[1] and L. R. Scrivner[2]
(Proc. Paper 1045)

FOREWORD

This paper is one of a group to be presented at the ASCE Symposium on Arch Dams, June, 1956, at Knoxville, Tennessee.

Since the last symposium on masonry dams (April, 1939), much progress has been made in the design and construction of arch dams and their appurtenances. This Symposium was planned to enable engineers concerned with arch dams to exchange their ideas and experiences for the benefit of all.

At this time, it is not known exactly how many papers will be printed from the Symposium. So far, twelve papers have been approved: "Arch Dams: Their Philosophy," by Andre Coyne (Proc. Paper 959); "Arch Dams: Trial Load Studies for Hungry Horse Dam," by R. E. Glover and Merlin D. Copen (Proc. Paper 960); "Arch Dams: Portuguese Experience with Overflow Arch Dams," by A. C. Xerez (Proc. Paper 990); "Arch Dams: Theory, Methods, and Details of Joint Grouting," by A. Warren Simonds (Proc. Paper 991); "Arch Dams: Santa Giustina Single-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 992); "Arch Dams: Measurements and Studies on Santa Giustina Dam," by Claudio Marcello (Proc. Paper 993); "Arch Dams: The Reno Di Lei Double-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 994); "Arch Dams: Isolato Double-Curvature Arch Dams," by Claudio Marcello (Proc. Paper 995); "Arch Dams: Rio Freddo Dam with Gravity Abutments and Cut-offs," by Claudio Marcello (Proc. Paper 996); "Arch Dams: Design and Observation of Arch Dams in Portugal," by M. Rocha, J. Laginha Serafim, and A. F. da Silveira (Proc. Paper 997); "Arch Dams: Development in Italy," by Carlo Semenza (Proc. Paper 1017); "Arch Dams: Design of the Kamishiiba Arch Dam," by C. C. Bonin and H. W. Stuber (Proc. Paper 1018); and "Arch Dams: Design and Construction of Ross Dam," by C. E. Shevling and L. R. Serivner (Proc. Paper 1045).

As other papers are approved, they will be published in the Proceedings.

Note: Discussion open until January 1, 1957. Paper 1045 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 82, No. PO 4, August, 1956.

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The interested reader should watch for these papers in following issues of the Journal of the Power Division.

INTRODUCTION

Ross Dam is the most important unit in the hydroelectric development of the Skagit River by the city of Seattle, Washington. It provides the main storage for the entire system which includes the Ross, Diablo, and Gorge Powerplants. These plants have a total generating capacity of 580,000 kw.

Diablo Lake, having a storage capacity of 90,000 acre-feet, and Gorge High Dam Reservoir, which has a storage of 6,000 acre-feet, serve to supply hydrostatic head for their respective plants and to accommodate the daily fluctuations caused by peaking at the Ross Plant.

Description

Ross Dam is located on the Skagit River approximately 100 miles northeast of Seattle, Washington. It is a variable radius arched structure which ranks as one of the world's highest arch dams. It rises 540 feet above foundation rock to an elevation 1615 feet above sea level and contains 905,000 cubic yards of concrete. At the crest, the dam has a length of 1,300 feet and an extrados radius of 600 feet. It varies in thickness, at the crown, from 208.5 feet at the base to 33 feet at the crest. It is constructed in 26 sections, separated by keyed contraction joints. Each section is 50 feet wide along the axis of the dam. During construction, the sections were raised systematically in a manner that provided for alternate low, intermediate, and high sections.

Ross Dam is unusual in some of its structural features. These include the provision for additional construction beyond the present stage to raise the dam to ultimate elevation 1733, a total height of 658 feet. This contemplated additional construction will require thickening the entire dam, from the base up, by adding to the downstream side where patterns of square and vertical keys were cast in the concrete to form a keyed bond between the old and the new concrete sections. (See photograph on next page.)

Other unusual features are the two hooded spillways, incorporated in the ends of the dam and designed to discharge 127,000 cubic feet per second. The hoods deflect the flow through the spillways, downward, to conform to the steep downstream face of the arch dam. Their design was determined from experimental scale models built in the Bureau of Reclamation Laboratories in Denver. The flows of water routed down the two spillways collide in midstream, partially dissipating their energy against each other. The spillway hoods are built of concrete, heavily reinforced with steel.

Purposes Served

At its present height, Ross Dam creates a storage reservoir of 1,405,000 acre-feet. Of this storage 1,023,000 acre-feet are usable for power generation. Thus, it serves to impound water from which the turbines of three powerplants, namely, Ross, Diablo and Gorge, extract power between

elevations 1600 (maximum) and 495. With Gorge High Dam scheduled to be completed early in 1959, every foot of this head between the forebay of the Ross Plant and the tail water of the Gorge Plant will be harnessed for power generation.

As required by the Corps of Engineers, Department of the Army, through the Federal Power Commission, the water level in Ross Lake is regulated in accordance with their prescribed rules for flood control. This provides valuable flood protection for the farm lands and communities of the lower Skagit River Valley.

History of Stepwise Construction

As a measure of economy and to match the anticipated increase in demands for power, a plan of stepwise construction was initiated for Ross Dam.

The first step was originally designed to be carried to elevation 1515 with the spillway crest at elevation 1500. This would provide a storage of 500,000 acre-feet for the Diablo and Gorge Powerplants. No power installation at Ross was planned for this step.

A contract was awarded in August 1937 for the construction of a portion of the First Step of the dam to elevation 1300. However, with money saved when excavation was not required to the depths estimated by the engineers, it was possible to continue placing concrete until elevation 1365 was reached. In addition, a temporary 15-foot timber crib was constructed on top of the concrete dam, raising its height to elevation 1380. This created a storage of 100,000 acre-feet for operation of the Diablo and Gorge Plants. This first-step construction required 330,000 cubic yards of concrete.

The Second Step, as it was then planned, required starting again by thickening the dam at the base and constructing it on a design which would permit carrying it to an ultimate height at elevation 1725. This sequence was necessary because at that time the plan was to construct the powerhouse on the face of the dam.

Studies made subsequent to the completion of the first contract indicated that it might be possible to carry the First Step to a greater height than the originally planned elevation 1515. This, together with constructing the powerhouse on the left bank downstream from the dam, instead of on the face of the dam, would make considerable power available without going to the expense of thickening the base for the high dam.

While this change in plan was being studied, the War Production Board started urging the city to proceed with increasing the height of the dam in order to aid the war effort. This the city agreed to do and a contract was awarded in February 1943 to raise the dam from elevation 1365 to elevation 1550. This was designated as the Second Step of Ross Dam, and would have required 456,900 cubic yards of concrete.

During construction of the Second Step, the consulting engineers, upon conclusion of their studies and calculations, determined that the dam could be constructed to elevation 1615 on the original First Step base. The Federal Power Commission approved construction of the dam to this elevation, its present height. An important factor which influenced this approval was the high strength of the concrete which permitted a higher allowable unit stress in the structure.

With the changes in the original concept of the construction program, the

steps of construction for the dam became and remain, identified as follows:

The First Step was the first contract which brought the concrete to elevation 1365.

The Second Step raised the dam to elevation 1550.

The Third Step is the dam to its present height at elevation 1615.

The Ultimate Height is the plan for the future which will thicken the present structure and raise it to elevation 1733.

The First Step contract was awarded to the General-Shea-Columbia Construction Company in August 1937 and completed in 1940.

The Second Step contract was awarded to General-Shea-Morrison Construction Company in February 1943. Before the completion of this contract, it became evident that to meet the growing demand for electrical energy, the Third Step of the dam should be constructed without delay. In order to save the time and expense of an extra cleanup and of moving out one construction plant and moving in another, the construction of the Third Step was added to the Second Step contract by change order. Thus, the Second Step was never completed as planned. The construction carried through in one continuous operation from the beginning of the Second Step to the completion of the Third Step.

The Third Step contract was completed in June 1949.

Stress Studies

The trial-load studies which were made for the design of Ross Dam were carried out according to the Bureau of Reclamation procedure (1) for analyzing arch dams by the trial-load method.

An analysis was made of the proposed Third Step construction for a water-surface elevation of 1635. The contraction joints were assumed to be grouted with the reservoir empty and the concrete at mean (2) annual temperature, except the temperatures were assumed to be 5° and 2° F above mean annual at elevations 1200 and 1250, respectively. This analysis included effects of tangential shear and twist. The maximum computed principal stress was 1,114 psi at elevation 1200.

An analysis was also made of the proposed Second Step construction for a maximum water-surface elevation of 1548.8. Cooling studies (see section on Cooling Studies) were made in connection with this analysis and it was assumed that the contraction joints would be grouted with the water surface at elevation 1295. This study indicated a maximum principal stress of 752 psi at elevation 1200. This analysis also included effects of tangential shear and twist.

When it was decided to place the concrete for the Second and Third Steps as a continuous operation, a limit was set on the height of dam so the following allowable stresses would not be exceeded:

For normal water surface and neglecting earthquake:

Principal stress, 1,000 psi

Shear stress at concrete-to-rock contact plane, 350 psi

For normal water surface plus earthquake effects or for maximum flood conditions without earthquake:

Principal stress, 1,250 psi

Shear stress at concrete-to-rock contact plane, 425 psi

The stresses all to be determined by a complete[3] trial-load analysis.

A plan view of the dam is shown on Figure 1; representative arch elements are shown as contours on the figure. A section at the line of centers (crown cantilever) is also shown.

The trial-load stress study of the dam, as constructed, was based on the following constants, assumptions, and loading conditions:

1. Top of dam, elevation 1615
2. Base at maximum section, elevation 1075
3. Normal water surface, elevation 1600
4. Earthquake effects neglected
5. Sustained modulus of elasticity of concrete and abutment rock, 3,000,000 psi
6. Shear modulus of concrete, 1,250,000 psi, reduced to 1,000,000 psi in calculating the detrusions due to radial shears to allow for the non-linear distribution of shearing stresses between the upstream and downstream faces of the dam
7. Poisson's ratio for concrete and rock, 0.20
8. Unit weight of concrete, 156 pounds per cubic foot
9. Coefficient of thermal expansion, 0.000,004,6 per degree Fahrenheit
10. The horizontal arch elements assumed to be symmetrical with radial abutments
11. Radial contraction joints in the original structure and in the new concrete assumed to be thoroughly grouted and satisfactory bond established between old and new concrete so the dam will act as a monolithic structure
12. Cantilever elements have radial sides 1 foot apart at the axis of the dam
13. Normal arch and cantilever stresses assumed to have a linear variation from upstream to downstream face
14. The concrete in the dam and the foundation rock assumed to have adequate strength to carry the imposed loads without exceeding their elastic limits

The effects on stress conditions in the dam due to the grouting and construction programs were included in the trial-load analysis, each stage of construction corresponding to a part of the analysis as follows:

Part 1

Contraction joints in the First Step construction grouted with the water surface at elevation 1285. All loads assumed to be carried by cantilever action. Stresses computed by gravity analysis at radial-sided cantilevers (see Figure 2 for results).

Part 2

Crest of dam raised to elevation 1500. Water surface raised to elevation 1354. Additional load carried by gravity action above elevation 1365, but divided between arches and cantilevers below elevation 1365 by means of a radial adjustment of deflections (see Figure 3).

Lift between elevations 1365 and 1400 grouted with reservoir water surface at elevation 1354.

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3. A complete trial-load analysis is used to mean that the effects of tangential shear and twist are included.

Part 3

Crest of dam raised to elevation 1615. Water surface reduced to elevation 1295. Additional load carried by gravity action above elevation 1400, but divided between arches and cantilevers below elevation 1400 by means of a radial adjustment of deflections (see Figure 4).

Portion of dam between elevations 1400 and 1615 grouted with the water surface at elevation 1295.

Part 4

Water surface raised to elevation 1600. All loads above elevation 1567 assumed to be carried by gravity action (due to through spillways). Load below elevation 1567 divided between arch and cantilever elements, by means of a complete adjustment, including the effects of temperature change from average concrete temperatures at the time the contraction joints were grouted to the minimum cyclic temperature (see Columns 2 and 3 of Table 1).

Stresses in the completed structure were obtained by super-imposing stresses calculated from the analyses made for the conditions stated in Parts 1, 2, 3, and 4 (see Figures 5 and 6). These stresses include effects of tangential shear, twist, and Poisson's ratio. In the complete analysis, deflections due to Poisson's ratio, were computed and included in the study at the end of the second radial adjustment.

Discussion of Stresses

The stresses tabulated on the figures are given in pounds per square inch (psi) and the following symbols are used:

E = stress at extrados of arch
I = stress at intrados of arch
U = stress at upstream face of cantilever
D = stress at downstream face of cantilever
\+ indicates compression
Crown of arch indicated by small square

Sections shown to the right of the stress drawings indicate the extent of construction and grouting as well as the reservoir water-surface elevations. The grouted portion of the dam is indicated by cross-hatching. Concrete which was included in a previous adjustment is indicated by hatching; new concrete has no hatch mark.

The load distribution is also shown at the crown cantilever with the cantilever load indicated by hatching. Load plotted to the right of the zero line indicates negative cantilever load, at which point the arch carries the external load plus an amount equal to the cantilever load.

Vertical cantilever siresses, due to loading Part 1, are shown on a profile of the dam (see Figure 2). The maximum computed stress of 382 psi occurs at the downstream face of the base of the crown cantilever.

Arch stresses normal to radial planes and vertical cantilever stresses for Part 2 loading are shown on Figure 3. The maximum computed stress amounts to 457 psi at the base of the dam. This would amount to 604 psi parallel to the downstream face.

Similarly, stresses through Part 3 loading are shown on Figure 4. The maximum computed stress is almost identical to the corresponding value for Part 2, as given in preceding paragraph. A tensile stress of 85 psi is

indicated at the crown intrados of the arch element at elevation 1350. If a crack were developed here, instead of the tensile stress, it would have very little effect on the final distribution of load and stress in the structure. The stresses discussed above are based on adjustment of radial deflections at points of intersection of arch and cantilever elements.

Arch and cantilever stresses, for loading through Part 4, are given on the profile of Figure 5. These stresses are based on a complete analysis and included effects of Poisson's ratio and temperature changes listed in Column (4) of Table 1. Principal stresses are shown graphically on Figure 6, and numerically in Table 2. The direct and principal stresses are assumed to act parallel to the faces of the dam.

The maximum computed arch stress is 615 psi and occurs at the crown extrados at elevation 1540. Stresses were computed at elevation 1540, but were omitted from the figures to make the figures clearer. The maximum arch stress at the abutments is 433 psi at the arch intrados at elevation 1250. There were no tensile stresses in the arch elements for this loading condition.

The maximum cantilever stress, 772 psi, occurs at the base of Cantilever E at downstream face. The stress at the base of the crown cantilever is 709 psi. There were no tensile cantilever stresses for this loading condition. However, with reservoir partially full, at a time when concrete temperatures are at or near maximum values, as given in Column (3) of Table 1, tensile stresses of moderate amount would occur at the downstream face of some cantilevers. This is not considered critical, particularly since the tensile area would be under compression during maximum loading of the structure.

The maximum computed principal stress is 800 psi, 80 percent of the allowable design value of 1,000 psi. All principal stresses were compressive except for a value of 20 psi tension at the downstream face at elevation 1500.

The maximum horizontal shearing stress on the concrete-to-rock abutment plane is 340 psi (see the final column of Table 2). This is 97 percent of the design value of 350 psi.

The stresses discussed above include the effects of average concrete temperature, but do not reflect the effects of temperature gradients due to seasonal or daily variations. It is apparent that daily and seasonal changes in temperature will have considerable effect on the stress condition near the surface of the dam. The surface stress will fluctuate over quite a wide range, but will be relieved somewhat by plastic flow during periods when the surface of the structure is hot and by surface checking during extremely cold weather. However, the condition of a drop in mean concrete temperature combined with maximum loading is as severe as any to which the dam may be subjected.

Temperature Studies

Temperature studies were based on temperature records at Diablo Dam, and at Concrete (formerly Baker), Washington. From these data, it was estimated that the mean annual air temperature would be close to 48°-50° F. Reservoir water and cooling water temperatures were estimated from observed water temperatures in the existing Ross Lake, and in Diablo Lake and Happy Creek. Studies were then made to find the range of mean concrete temperatures at the several arch elevations. The first study was solely for Second-step construction, and was later extended to cover construction of the

combined Second and Third steps. The results of the latter temperature study were as shown in Table 1.

The trial-load analysis of the Second Step dam was made using the above data and taking into account the desired closing temperature (40° F) and the temperature rise to the minimum temperature condition. A complete trial-load analysis was later made for the dam, as constructed, using the actual closure temperatures experienced at the several arch elevations.

Artificial cooling of the concrete was required primarily for cooling the concrete to the desired temperature so that contraction joint grouting could be accomplished either during or immediately after the construction period. Additional benefits included the reduction of the maximum temperatures and the crack prevention measures obtained by reducing the temperature gradients near the faces of the dam and near the contact of the existing concrete with the new concrete. Temperature studies indicated that a cooling transition zone would be beneficial in the first lifts of the new concrete. The horizontal spacing of the cooling tubing was, therefore, 2-1/2 feet on top of the existing concrete and on top of the first 5-foot lift of concrete above the existing concrete, 3-1/2 feet on top of the second lift, 4-1/2 feet on top of the third lift, and 5-1/2 feet on the fourth lift. This last spacing was then continued to the top of the dam.

Studies were also made as to the expected temperatures in the concrete at the time of secondary cooling and the time required to lower the temperature of the concrete to the required temperature in time for contraction joint grouting.

Method of Joining New and Old Construction

The old concrete was thoroughly cleaned and chipped to obtain good bond between the new and old concrete. Where necessary wet sandblasting was used to remove all laitance, efflorescence, grease, mud, debris, pools of water, and loose or defective concrete. The surface of the old concrete was thoroughly soaked with water for several days prior to placing new concrete. Just prior to placing new concrete, 1/2-inch of mortar was placed over the surface and thoroughly broomed into the surface.

The first new concrete placed was provided with cooling coils at reduced spacing to limit the temperature gradient at the contact plane between old and new concrete. (See paragraph on Temperature Studies.)

Water Stop Details

Copper water stops were installed across each vertical contraction joint at the upstream face of the dam. These consisted of 16-gage, 48-ounce copper strips 15 inches wide. The center 3 inches of the strip was bent double and projected 1-1/4 inches normal to the stop in the contraction joint to absorb the expansion at the contraction joint. Anchorage at the outer two edges of the strip was provided when embedded in the concrete by cutting the copper every 12 inches, 1 inch deep, and bending alternately to the right and left normal to the stop.

Horizontal water stops 10 inches wide of 16-gage, 48-ounce-per-square-foot copper strips were embedded in the old concrete when placed, and at top of each horizontal lift in new concrete. The water stops were located 9 inches from the upstream face of the dam.

The horizontal stops were brazed to vertical stops in each contraction joint to make a continuous water seal.

Abutment Preparation

The abutments were excavated to sound rock free from injurious seams or defects. Care was exercised to prevent shattering of the rock outside of the limits of the excavation. Blasting was very carefully controlled to preserve the sound rock and to prevent injury to the structure.

The foundation was prepared by removing all loose, weathered, disintegrated and shattered rock and shaping the abutment for the placing of concrete. The surfaces were left rough so that a good bond would be secured with the newly placed concrete. The rock surfaces were cleaned by brooms, hammers, picks, streams of water, air, and wet sandblasting. All water was removed from the surface of the foundation before concrete was placed thereon. The foundation was thoroughly grouted and drained.

Cooling and Grouting Operations

The concrete of the First Step was placed without provision for artificial cooling. It was grouted in the Spring of 1946, thereby taking advantage of natural cooling during the winter months. Mean concrete temperatures at time contraction joints were grouted are given in Column (1) of Table 1.

Concrete cooling operations, in Second and Third Step concrete, included an initial cooling period and a secondary cooling period. The initial cooling period was to reduce the maximum temperatures and to obtain a more beneficial temperature differential between the interior and the face of the dam. The initial cooling period varied from 6 to 12 days, depending on the time of year. Secondary cooling to lower the concrete temperatures to the desired grouting temperatures required approximately 45 to 50 days, depending on exposure conditions and the temperature of the available cooling water.

Secondary cooling was started about December 1, 1946, but was stopped about 2 weeks later due to extremely cold weather which froze the exposed header systems. Cooling was resumed about February 1, 1947, and temperatures were lowered to approximately 40° F. However, due to an early runoff and the resulting high reservoir level, grouting was interrupted and only a part of the grouting planned for the Spring of 1947 was completed. Because of the difficulties experienced in the 1946-47 winter season, a refrigerating plant was obtained and installed in the Fall of 1947 to assure that the remainder of the secondary cooling would be completed in the Spring of 1948. The plant consisted of fourteen 50-ton commercial-type units which supplied 29° F brine to the cooling coils. Secondary cooling was started near the end of January 1948, and was completed to the top of the dam in March 1948.

Trial-load analyses indicated that it would be desirable to grout the contraction joints when the reservoir elevation was at or near elevation 1295. Because of the early and large river runoff in the Spring of 1947, however, the contraction joint grouting program was speeded up and grouting was started in the abutment joints when the reservoir was about elevation 1420 and dropping. The reservoir gradually dropped to elevation 1354 and then started rising. Abutment joints were grouted to elevation 1450 during this time but the central part of the dam, from Joints 10 to 18, inclusive, was only

completed to elevation 1400. In the Spring of 1948, all contraction joints were grouted to elevation 1615 with the reservoir between elevations 1288 and 1301.

A summary of contraction joint grouting operations is given in Table 3. In the 1946 operations, 1,748 sacks of cement were used in grouting 204,770 square feet of joint area (keyways neglected). The average joint area covered for each sack of cement was 117 square feet. In the 1947 operations, 846 sacks of cement filled 95,393 square feet of joint area, or 113 square feet per sack of cement injected. In the 1948 operations, 2,139 sacks of cement filled 159,045 square feet of joint area, or 74 square feet per sack of cement injected.

ACKNOWLEDGMENT

The authors gratefully acknowledge the valuable assistance and contributions by members of the Bureau of Reclamation, and of the Seattle Department of Lighting.

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Table 1

MEAN CONCRETE TEMPERATURES

Temperature in degrees Fahrenheit

Elevation	At time of joint closure	Minimum *	Maximum *	Change ** included
	(1)	(2)	(3)	(4)
1200	48.0	47.5	51.0	-0.5
1250	47.0	47.5	51.5	±0.5
1300	47.0	47.0	52.0	0
1350	44.0	47.0	52.5	±3.0
1400	41.0	46.5	54.0	±5.5
1450	39.1	46.5	55.5	±7.4
1500	35.9	45.5	57.0	±9.6
1567	34.9	43.0	60.0	±8.1

* Due to annual cyclic variation

** Minus sign indicates temperature drop. Plus sign a temperature rise.

Table 2

PRINCIPAL STRESSES AND MAXIMUM SHEAR* STRESS

Elevation	Upstream face P_1	Upstream face P_2	Upstream face a	Downstream face P_1	Downstream face P2	Downstream face a	Max shear*
1567	± 51	±199	9° 50'	±293	± 28	76° 43'	17
1500	± 75	±282	15° 10'	±309	- 20	50° 04'	42
1450	±104	±139	61° 20'	±453	± 39	64° 07'	164
1400	± 89	±144	74° 05'	±446	± 61	59° 21'	202
1350	± 56	±130	70° 33'	±483	± 75	48° 20'	228
1300	± 66	±215	83° 39'	±455	± 17	59° 29'	216
1250	± 98	±243	32° 13'	±636	±216	45° 52'	267
1200	± 27	±155	30° 44'	±800	±164	20° 52'	340
1075	±151	0	0° 00'	±709	0	0° 00'	244

* On concrete-to-rock contact planes

Table 3

GROUTING PROCEDURES--ROSS DAM

Grouting operation	Date grouted	Grouted to elevation	Joints grouted	Joint Area sq. ft	Cement placed (sacks)
1	2-4-46	1200	14 & 15	25,073	294
2	3-6-46	1250	11-15	30,539	184
3	3-11-46	1300	14-19	32,705	277
4	3-14-46	1365	15-19	27,240	208
5	3-18-46	1300	8-13	36,239	316
6	3-21-46	1365	9-14	31,806	227
7	3-27-46	1365	7, 8 & 20	21,168	242
		Subtotal 1946 operations		204,770	1,748
8	3-13-47	1400	19-21	12,197	145
9	3-19-47	1400	7-9	9,883	120
10	3-27-47	1400	21	4,181	55
11	4-16-47	1400	10-18	20,537	160
12	4-21-47	1450	19-22	21,195	146
13	4-23-47	1450	5-9	27,400	220
		Subtotal 1947 operations		95,393	846
14	2-20-48	1450	10-18	24,131	215
15	2-22-48	1500	18-23	14,984	87
16	2-29-48	1500	4-9	16,279	204
17	3- 2-48	1500	21	2,768	27
18	3- 4-48	1500	10-17	18,235	342

Table 3 (continued)

GROUTING PROCEDURES--ROSS DAM

Grouting operation	Date grouted	Grouted to elevation	Joints grouted	Joint Area sq. ft	Cement placed (sacks)
19	3- 6-48	1550	18-24	12,646	205
20	3- 8-48	1550	4-10	15,471	170
21	3- 9-48	1550	12, 14, 16-17	—	47
22	3-11-48	1550	11-17	14,464	240
23	3-12-48	1600	18-24	8,870	142
24	3-13-48	1600	2-10	11,522	158
25	3-15-48	1600	11-17	12,283	194
26	3-16-48	1614	1-25	7,392	108
		Subtotals 1948 operations		159,045	2,139
		Total all joint grouting		459,208	4,733

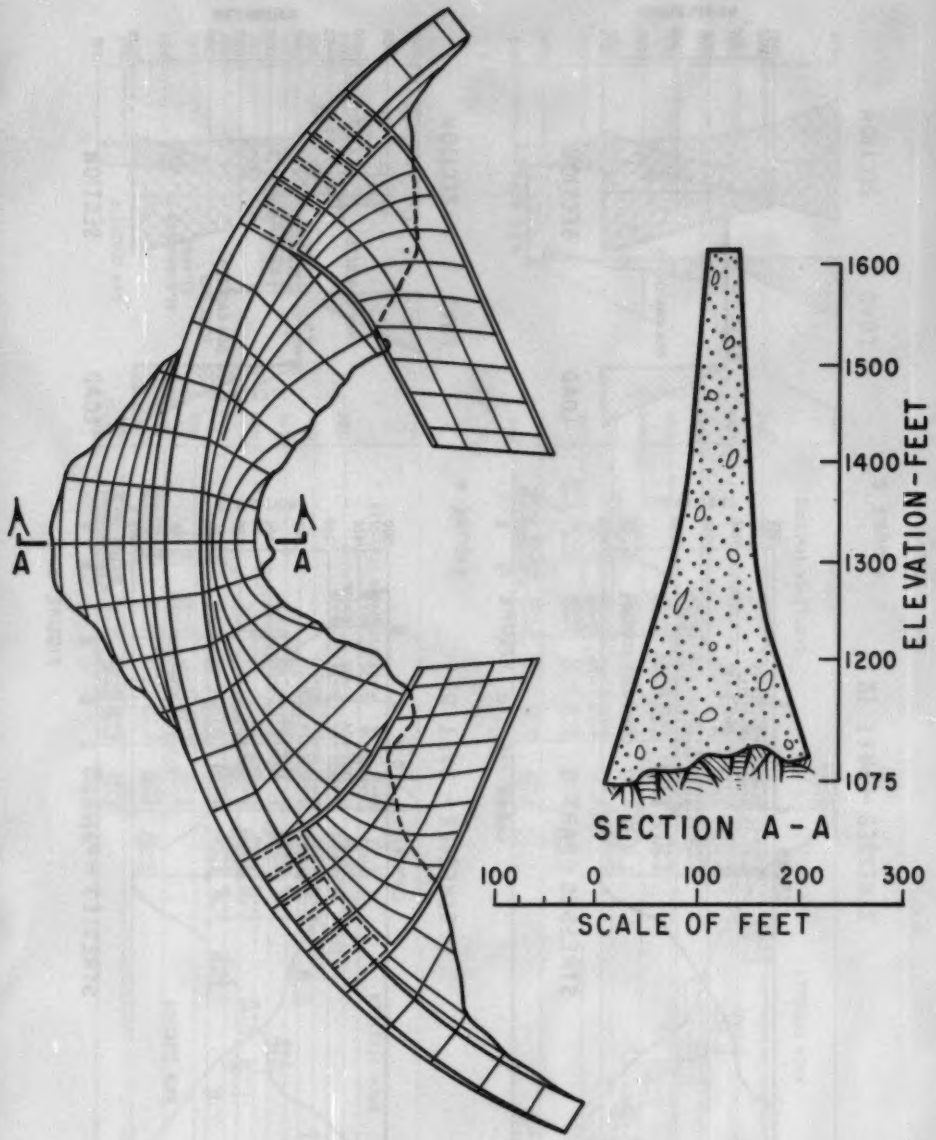
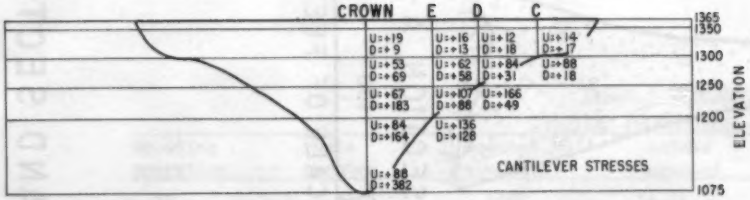
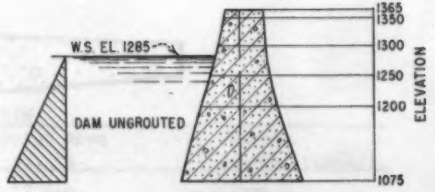


FIG. 1 - ROSS DAM - PLAN AND SECTION

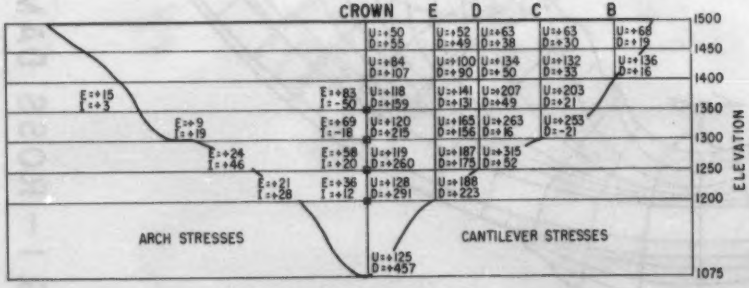


STRESSES - PART I



LOAD SECTION

FIGURE 2



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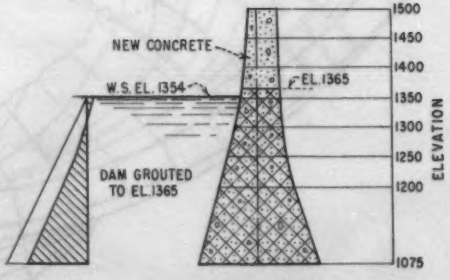
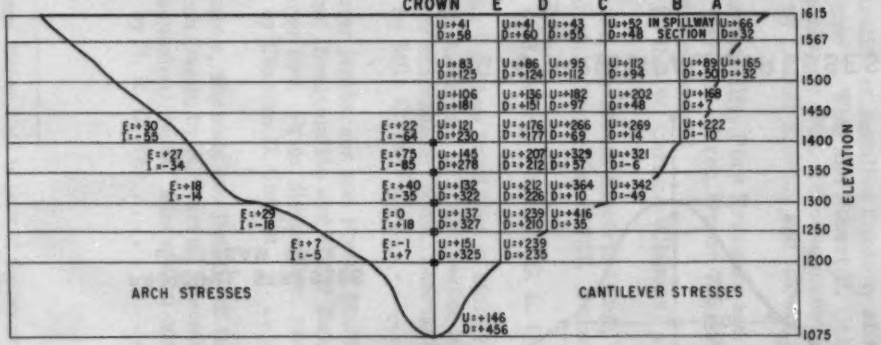


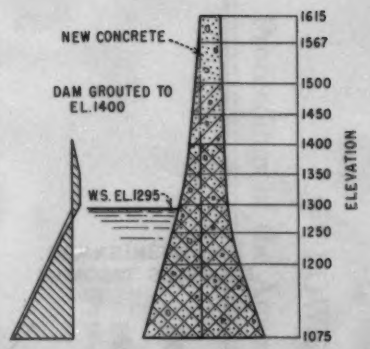
LOAD SECTION

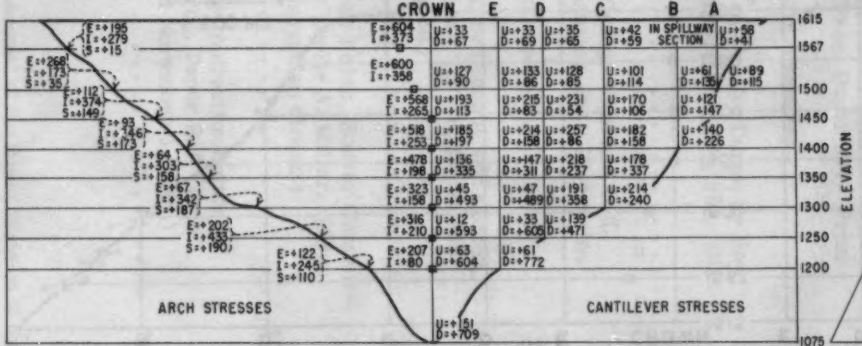
FIGURE 3



STRESSES – PART III

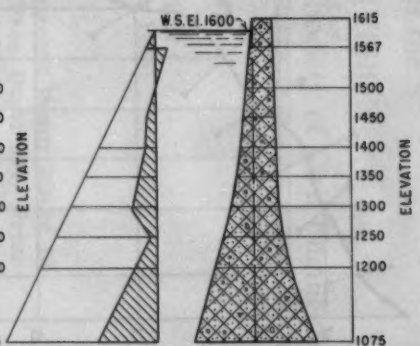
FIGURE 4





STRESSES – PART IV

FIGURE 5



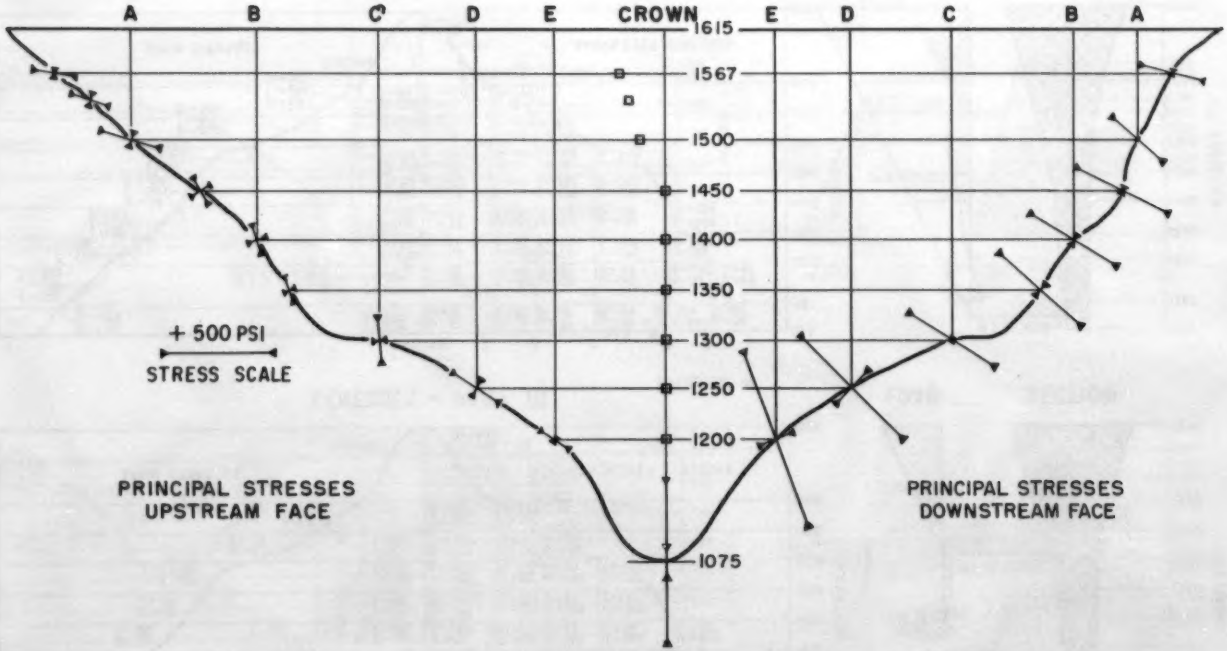


FIG. 6 PRINCIPAL STRESSES

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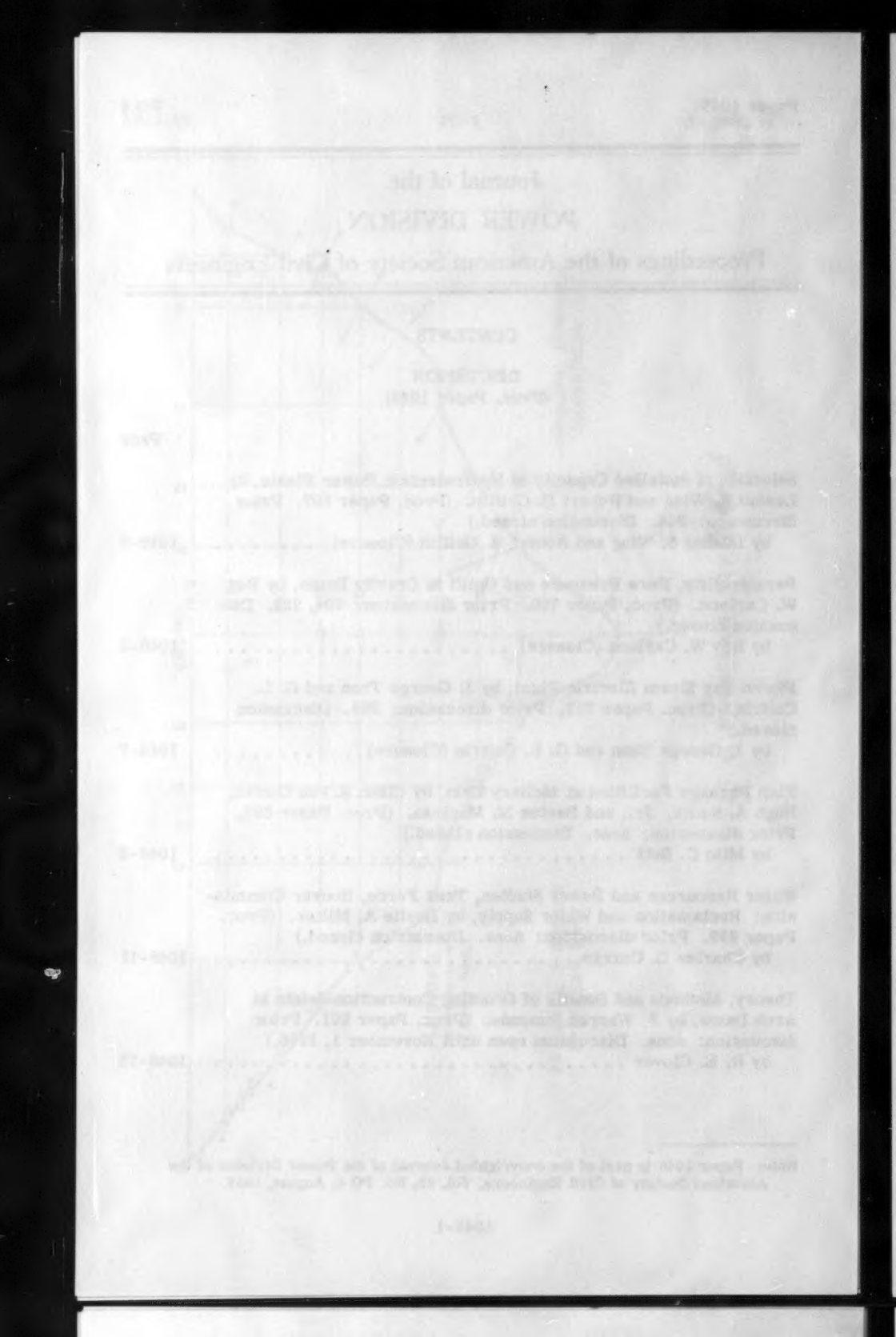

Discussion of "SELECTION OF INSTALLED CAPACITY AT HYDROELECTRIC POWER PLANTS"

by Lesher S. Wing and Robert H. Griffin
(Proc. Paper 697)

LESHER S. WING[1] AND ROBERT H. GRIFFIN.[2]—The writers wish to thank Mr. Kraft for his discussion and particularly for his presentation of the factors involved in reserve capacity. The writers believe that it is quite difficult to consider the question of reserve capacity in the studies outlined as a guide to the selection of ultimate installation for which a project should be designed. However, it is certainly true that the economic provision of reserve capacity is important and that it should be considered when determining the size, kind, and location of capacity additions currently being made to the system. We believe that reserves affect the timing of hydro capacity additions rather than the ultimate economic installation at a particular site.

The writers agree with Mr. Kraft in his statement that power must be evaluated at the load areas or load centers where it is used. The use of the term "at market" was intended to convey this concept. Mr. Kraft's clarification is appreciated.

In the writers' paper the proposed procedures are illustrated by using prime and secondary power and energy. This concept was adopted for three reasons. First, in the planning and design stages of a project it is frequently not possible to predict accurately the system load characteristics and the other power sources with which the project will be combined say fifteen or more years hence. In the case of federal projects the type of load to be served during even the initial period is frequently uncertain. Yet the initial layout must be such that it will provide space for the future units that may reasonably become economic. In all such cases, the writers believe the proper approach is to use the project's prime output as the starting point.

Second, the firm power output of a hydro project is variable and less predictable than the prime. It depends upon current load characteristics; the amount of hydro storage available; how storage is operated; and the system hydro-steam ratio.

Third, ultimately all hydro energy—both prime and secondary—should be usable either as firm energy or as steam replacement. When this period is reached, the distinction between firm and prime is unimportant, so long as

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there is sufficient firm (or prime) to permit the plant's capacity to be dependable on the system load.

The proper hydro installation at any particular time is, in the writers' opinion, very directly related to amount and kind of reserves needed, the most economic source of reactive kva, and other special, although frequently temporary, operating conditions that must be met. The writers did not intend to limit the suggested procedures to the use of prime, and this is pointed out in Footnote 7 which states, in part, * * * * "it is usually desirable to use seasonal, and in some cases monthly curves in making the analyses described in this paper." The use of firm or prime energy will be governed primarily by the data available, and whether the planning is for an addition to an existing plant, or the initial layout and design of a project.

It is believed that a properly prepared power period duration curve or separate duration curves for each month will adequately reflect, and allow evaluations of, the variations in power output described by Mr. Kraft under "Average Usable Power Supply." The hydro-steam ratio at which all wet-year hydro energy becomes usable is subject to wide variation depending on hydrologic, load, and other conditions. The writers suggest that in many systems all available hydro energy will be utilized when the hydro-steam ratio reaches 50-50.

The writers agree with Mr. Kraft that many factors are involved in a determination of economic installed capacity at hydroelectric power plants and that all should be considered to avoid error. It is hoped that the original paper and Mr. Kraft's able discussion will be of some value in attaining this desirable objective.

Discussion of "PERMEABILITY, PORE PRESSURE AND UPLIFT IN GRAVITY DAMS"

by Roy W. Carlson
(Proc. Paper 700)

ROY W. CARLSON,* A.M. ASCE.—After studying the discussions by J. L. Serafim and Serge Leliavsky, the writer wishes to revise his conclusions regarding pore area subject to uplift. Conclusion "3" in the paper would then read as follows:

3. The maximum pore area in concrete subject to pore pressure can not be determined from its effect on strains over customary gage lengths; strain measurements can at best yield only some sort of average area. The only known and defensible method for determining the one, maximum pore area which exists in a specific concrete is that of testing to failure, as was done by Leliavsky.[1] An approximate estimate can be made of the maximum area, however, by studying idealized models which have all of the essential properties of concrete.

The discussion of J. L. Serafim shows a good understanding of the subject of uplift but the writer can not agree that the effective areas he determined from strain measurements were the MAXIMUM areas of pores. The writer remains convinced that strain measurements can not reveal the "worst" area. While it is true that strains in concrete subjected to pore pressure are affected most by regions of large pore area, they nevertheless represent a weighted average of all portions of the concrete within the gage length. There appears no logical reason why strains over a considerable gage length should reveal the one largest pore area in a specimen. And the writer agrees with Mr. Leliavsky that the "weakest section" is the one which should govern.

Mr. Leliavsky points out the danger in oversimplification when considering idealized models, and the writer agrees. However, much can be learned from such models IF the model possesses all of the essential, measureable properties of the concrete. Mr. Leliavsky asks, "what happens with the uplift pressure if we shift all of the imaginary solids of the 'ideal' pattern so as to form a number of vertical columns, and all of the voids so as to form vertical cells?" The answer to this is that you no longer have anything simulating concrete. The writer insists that any idealized model must have all of the essential properties of the concrete itself. If the model has vertical columns of solids, the modulus of elasticity will be too high, since the modulus of elasticity of the solids is usually several times that of the concrete. Furthermore, the properties of the model will be far different in different directions. The model postulated in the paper, on the other hand, maintains all of the essential, measureable properties of the concrete. Not only are the moduli of

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1. Serge Leliavsky Bey, "Experiments on Effective Uplift Area in Gravity Dams," Trans. A.S.C.E., Vol. 112, 1947, p. 444.

elasticity and poissons ratios of both solids and composite concrete preserved, but the porosity is correct also. And the properties are the same in each of the three principal directions of measurement.

In support of the conclusion shown in the paper that concrete is relatively impermeable, the writer maintains that the leakage which occurs through dams does not occur through sound concrete. After inspecting scores of dams in service, many of which leaked badly, he has yet to see evidence of water percolating through sound concrete. It is true that concrete varies greatly from point to point, but in a thick dam there are not likely to be any long percolation paths (still considering only the sound concrete), and the average permeability should be very nearly the governing one. Percolation of water through dams is a serious problem, but the percolation is likely to be through cracks and joints. This view is supported by the observed fact that recent trends toward lower cement contents in dams has led to lesser leakage rather than greater.

Mr. Bryant Mather's contribution of the results of permeability tests on lean concretes up to 5-years age is greatly appreciated.

Discussion of
"MORRO BAY STEAM ELECTRIC PLANT"

by J. George Thon and G. L. Coltrin
(Proc. Paper 737)

J. GEORGE THON,[1] M. ASCE, AND G. L. COLTRIN,[2] A.M. ASCE.—Mr. Woodruff expressed the opinion that the selection of the seismic factor of C = 0.13 for the design of the 450 feet high concrete stack was somewhat conservative. As explained by the writers, the factor of 0.13 was selected because of the desire to have the bending moments at all sections at least as large as those produced by a constant acceleration equal to 20 percent of gravity. The latter criterion has been commonly used in California when designing structures of the importance of a central steam electric station, where interruption of services might not only cause substantial financial loss but also widespread public inconvenience.

Mr. Field asked for the reasons for selecting a piled foundation under the stack while a floating mat foundation was used under the main building. There were two reasons for this decision:

1) The unit soil pressure under the buildings was reduced by 1.6 K/sq.ft. by removal of 16 feet of soil and the construction of a basement. The foundation for the stack was not readily adaptable to such a solution. On the other hand, in order to keep unit pressures within allowable limits would have required an excessively large mat.
2) The layer of weak to moderately firm clayey soils at the south end of the plant and indicated on Fig. 7 was proven to be of variable thickness. The consolidation of this stratum might have caused uneven settlements, a most undesirable feature in a 450 feet high slender structure.

Mr. Moore pointed out the value of the settlement records. It should be added that settlement readings were continued and that subsidence of points 2 and 5 on Fig. 7 ceased at about 1-3/4 inches. Points 8 and 13 came down to 1-1/2 inches and no further movement has taken place. These observations were taken at monthly intervals, the last being in February 1956.

Mr. Jordy suggested that it might have been structurally sounder if a constant seismic coefficient of 0.20 were used for the bottom one-fifth of the stack, and 0.13 or 0.15 were adopted for the upper four-fifths and distributed according to the 1953 A.C.I. Code. The writers considered a similar solution, however, after considerable thought, decided against such an expediency which would have failed to comply with either the A.S.C.E. or the A.C.I. Codes.

The authors wish to express their gratitude to the discussers for their valuable contributions. Their comments have clearly enhanced the value of the original paper.

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2. Senior Civ. Engr., Pacific Gas and Electric Co., San Francisco, Calif.

Discussion of
"FISH PASSAGE FACILITIES AT McNARY DAM"

by Glenn H. Von Gunten, Hugh A. Smith, Jr., and Berton M. Maclean
(Proc. Paper 895)

MILO C. BELL.[1]—The authors have made a factual approach to fishway design problems at McNary Dam so have portrayed neither the imaginative design which went into the creation of the structures nor the intricate design which was required to solve many of the operating problems of the fish facilities systems.

In considering the elements of this paper it must be realized that design engineers are not usually intimately connected with the development of formal structures tailored to the requirements for preserving living animals, in this case, fish. Usually operational considerations, costs or structural safety indicate the major direction of the design activity. In the case of the fishways, whose design has been described, the biological requirements of the fish were given the first consideration and this factor should be kept in mind when reviewing the sections of the paper and the statements on design criteria. The numbers of fish presenting themselves for passage at the dam are given in the paper and it is their timing and total numbers in a given day which set much of the space room requirements in the described structures.

These factors are discussed on pages 3, 4 and 5 and warrant careful review before proceeding with the more descriptive material on the major structures. It should be noted by those reading the paper that they have before them the details of the largest fishway system now in operation. The authors have rightfully pointed out that for those who may wish to interpret the design criteria it would be essential that they use judgment if they wish to use the same criteria elsewhere.

Many interesting engineering problems confronted the designers even in the matter of passage of water over weir crests. This interesting facet has been covered by the authors on pages 7 and 8 in the description of the fish ladder. Here it was found that, while weirs are widely used for the measurement of water, the shape of the weir has presented new problems to the designers of long fishways. To prevent fluctuations in flows an interesting series of tilting weirs was developed.

Also worthy of attention by the readers is the energy breakdown in the large quantities of water which are distributed through gratings at velocities of 1/4 foot per second and distributed uniformly over large areas. Such has been accomplished in the McNary fishway systems. This was not done, however, without the use of closely tailored models. In fact, many of the fishway features required detailed model studies to insure their proper function. These features are described throughout the paper in the sections covering the auxiliary water supply systems.

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1. Technical Coordinator, State Dept. of Fisheries, Seattle, Wash.

Fish Lock:

The Washington shore fish lock described on page 11, which is of sufficient size to pass a small craft, has required many innovations in lock design, particularly to balance the flows and to permit the ingress and egress of fish smoothly. The pressure lock is unique in this country, although it has counterparts elsewhere.

Another problem confronting the designers was the distribution of water in the three large conduits from the junction pool area on the Oregon shore fishway. This pool is described on page 16.

Beginning on page 20 is a description of the temporary fish passage facilities. Here again the reader should realize that as much care was given to the placement of these structures and their operation as to the placement of the permanent facilities. The construction period of this major dam was of sufficient duration to have endangered seriously the fish runs, had not temporary facilities been installed at the various construction phases. These fishways in themselves were of greater magnitude than many existing permanent systems.

The paper briefly points out the fact that the design of the spillway gates took into consideration the possible passage of downstream migrants. The brevity of this statement might mislead the reader into believing that the downstream migrant problem is not of such major importance as that of the upstream migrants. This is not the case and the reader must carefully evaluate the third paragraph of page 2 to get the proper impact on dam design of such considerations.

In conclusion, to those engineers who plan major facilities in fish producing streams the writer would recommend that they review carefully this most informative paper. I could not suggest a better case for the need of reviewing this paper than that presented by the authors in paragraph 4 of page 2 of the paper.

Discussion of "WATER RESOURCES AND POWER STUDIES, TASK FORCE, HOOVER COMMISSION: RECLAMATION AND WATER SUPPLY"

by Leslie A. Miller
(Proc. Paper 899)

CHARLES D. CURRAN,[1] M. ASCE.—Governor Miller's paper is an appropriate vehicle to bring to focus current Governmental activities and their relationship to the whole problem of the need for agricultural land. On April 11, 1956, a few weeks after the printed paper was available, the Upper Colorado River development was authorized as a Federal project and on April 16, the President, vetoing a farm bill, again requested the establishment of a Federal soil bank. Consideration of the evidence presented in connection with these two pieces of legislation and of the parts of the Task Force report pertinent to the agricultural land activities of the Federal Government, indicate not only that the overall program is confused but, equally important, that adequate data upon which to base a sound program are not available.

There appears to be a tendency to try to isolate the reclamation program as something apart from the national crop land situation. Figures reported by the Bureau of Reclamation show that 6,125,766 acres were under irrigation in 1954 on Federal projects. An additional 1,106,014 acres of irrigable land are on the Federally developed projects. However, something less than half of the total has been brought into cultivation from full aridity by the Federal Government. The balance originally developed without Federal aid has been brought supplemental water under the reclamation program. According to a recent statement of the Under Secretary of Interior, there are approximately 29 million acres of irrigated land in the country. This means that almost 23 million acres have been irrigated wholly by private enterprise. A press release of the Department of the Interior states that the Federal reclamation program will add something less than 50,000 acres during 1956. While this, in brief, is the irrigation-reclamation picture, it is only a small part of the crop land entirety.

Advocates of the Federal reclamation program have pointed out that approximately a million acres a year are being withdrawn from cultivation throughout the country to provide land for urban, highway, and airport developments. An apparent assumption is that the reclamation program must bear the full responsibility of replacing this land. Other figures on crop land, to say the least, raise questions.

The most recent figures of the Bureau of the Census give 345 million as the total acreage harvested. In addition, there are some 134 million acres of good crop land on farms, about half of which is used for pasture and about half of which is neither harvested nor pastured. There is, then, a considerable reserve of crop land already on farms.

Figures of the Department of Agriculture show that approximately 21

1. Consultant, Chevy Chase, Md.

million more acres of fertile land can be reclaimed for farming by drainage and clearing.

The Chief of Army Engineers, in a report of March 27, 1952, addressed to a subcommittee of the House of Representatives' Committee on Public Works, stated that the flood control activities of his bureau have permitted an improvement in agricultural production that is equivalent to the reclamation of 8 million acres of land. This, he stated, is spread over a gross area of 43.6 million acres which form a part of going agricultural areas in private ownership.

Representatives of the Department of Agriculture's Soil Conservation Service have advised that the watershed protection and flood prevention programs being carried out by that agency, could add 25 million acres to the Nation's available crop land.

The Wall Street Journal of September 1, 1955 stated that during 1955, 11 million acres were taken out of production on top of a 20 million acre cut in 1953 and 1954. Thus, 31 million acres have been withdrawn from cultivation under the Federal crop control and price support programs during the three years just passed.

The soil bank program proposes withdrawal of some 40 million acres from cultivation at a cost of $1.2 billion annually.

The Upper Colorado River Project will ultimately add the equivalent of about 200,000 acres at a cost of $324 million.

Since these figures have come from different sources, there may possibly be overlap among them. Nonetheless, it is clear that the various Federal programs are without adequate, if any, coordination. This problem of farm land is more than an engineering problem. But, here again, the engineering profession has a responsibility to show that it does not sponsor construction as an end in itself. Good design, efficient use of materials, and stable structures to bring more land to agriculture are not sufficient. As a first step in resolving the confusion that the figures quoted above indicate, some one agency should coordinate and correlate data as to the probable future food and fibre demands, the kinds of crops, if any, requiring additional land, and the relative economy of bringing into production the different kinds of land.

The reclamation program is a part of the water resources issue but it is more importantly a part of the crop land problem.

Discussion of "THEORY, METHODS AND DETAILS OF GROUTING CONTRACTION JOINTS IN ARCH DAMS"

by A. Warren Simonds
(Proc. Paper 991)

R. E. GLOVER,[1] M. ASCE.—The Author describes grouting procedures used for establishing structural continuity in Arch Dams after the setting heat has been dissipated. It is important that the concrete in the dam should be brought to thermal stability with its surroundings before the grouting is done because a continued shrinkage, due to cooling after grouting has been done, will set up strains which impair the ability of the dam to carry the imposed loads in the most effective manner. In massive arch dams an embedded pipe cooling system may be needed to bring the concrete temperatures to a predetermined level, before grouting is done, because the time required to cool by natural means may be intolerably long. The times required for geometrically similar solids to lose a specified proportion of excess temperature to their surroundings, under similar conditions, is proportional to the squares of similar dimensions.[2] The significance of this rule may be illustrated by comparing the Gibson and Hoover dams. In a dam of the thickness of Gibson about 90 percent of its excess temperature, due to release of setting heat by the cement, will be lost to its surroundings in a year and a half. In the Hoover dam, which is about ten times as thick, a century and a half would be needed to lose the same proportion. This rule explains why some dams need to be artificially cooled and others do not and it also explains why embedded pipe systems are needed in large arch dams.

It is often stated that concrete cooling is used as a crack control measure and in a sense this is true, since a crack is the result of excessive tensile stress. The use of artificial refrigeration to obtain low placing temperatures has been effective as a means of reducing cracks but this type of cooling can not take the place of embedded pipe cooling in massive arch dams. This is because cooling in arch dams is primarily a stress control measure and the embedded pipe system is the only one which gives the builder a positive control of the concrete temperatures. This system of temperature control has been used successfully on many massive dams. Curves of cooling, as calculated for use at Hoover Dam, permit such systems to be planned effectively in advance of construction. Comparisons of observed and computed performance of embedded pipe cooling systems show that their performance can be calculated with a precision that exceeds the needs for effective planning and use.

All dams are subject to temperature changes due to seasonal changes of

1. Research Engr., Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo.
2. "Flow of Heat in Dams," by Robert E. Glover. Journal of the American Concrete Institute. Proceedings Vol. 31, Nov-Dec 1934, Page 113.

the external temperatures. In dams thicker than about 35 feet the lag of the mean concrete temperatures behind the seasonal temperatures is one eighth of a period of fluctuation. For the seasonal changes this lag is about a month and a half. If the winter temperatures come to their lowest point at about the middle of January then the lowest mean concrete temperatures may be expected about the first of March. For thinner dams the lag is less.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW) divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 861 is identified as 861 (SM1) which indicates that the paper is contained in issue 1 of the Journal of the Soil Mechanics and Foundations Division.

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AUGUST: 761(BD), 762(ST), 763(ST), 764(ST), 765(ST)[c], 766(CP), 767(CP), 768(CP), 769(CP), 770(CP), 771(EM), 772(EM), 773(SA), 774(EM), 775(EM), 776(EM)[c], 777(AT), 778(AT), 779(SA), 780(SA), 781(SA), 782(SA)[c], 783(HW), 784(HW), 785(CP), 786(ST).

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c. Discussion of several papers, grouped by Divisions.

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